Examination into the Reliability of Seconday Water Supplies

by

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Abstract

Secondary water supplies have been included within the standard due to their perceived increase in the reliability of the water supplied to the sprinkler system, however, the decision on which secondary water supply best fits a building or region has not been adequately researched. By deriving the availability of the water supply, the different solutions covered by the New Zealand Standards can be compared.

The methodology considers the major influences on the reliability of a water supply headworks as well as the infrastructure involved in the secondary water supplies. In order to assess the reliability of these systems, critical components within the system need to be identified.

Comprehensive data collected for all the critical components of the water supply are analysed to obtain a comparison of the reliability of secondary water supplies. A Monte Carlo simulation is then used to generate random failures. These failure can be used to examine the reliability of the water supply.

The probabilities found during this examination confirm the additional reliability found in dual supply. It also shows that there are large variations in the reliability of the supply depending on the source of the town's mains supply.

Current literature describing the reliability of water supplies is examined and a case study of the Adelaide headworks is conducted showing the compatibility of the calculated values to those found within the literature.

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Chapter 1. Introduction

1.1 Overview

This report assesses the reliability of the water supply to sprinkler systems and aims to provide recommendations on the effectiveness of various water supplies to enhance the operability of the supply.

The supply of water to a sprinkler system requires at least one reliable source defined as the primary source. Where a building presents a higher risk or where it is difficult for fire service to control fires due to the size of the building an additional water supply or secondary supply operating in tandem with the primary supply is required under the current New Zealand sprinkler standard (NZS 4541:2007) for commercial buildings. These additional water supplies are provided to achieve a higher level of safety through increased reliability of the water supply feeding the fire sprinkler systems.

The New Zealand standard divides the design of these supplies into a number of classes. The aim of this is to provide sufficient capacity and a stable level of water pressure from a single supply. However, the New Zealand standard may also require the provision of an alternative source of water to increase reliability.

Within these secondary water supply classes, special consideration is given to areas prone to earthquakes. A typical post-earthquake scenario involves damage to all the major infrastructure of the water supply network. This historical occurrence of widespread damage to the water supplies coupled with the assumed high rate of fire occurring post-earthquake leads to an assessment of the system reliability when exposed to an earthquake to be considered along with the water supplies' operational reliability.

The reliability of the water supplies is dependent on the source of the supply. There are a number of alternative sources that can be used as a secondary supply to achieve a design compliant with the New Zealand sprinkler standard (NZ4541 2007). Some of the alternatives to achieve this are listed below with more discussion on the alternatives in chapter 2 and the full requirements of the sprinkler standard in Appendix A.

- Additional town's main which can be boosted or pump driven.
- Elevated, gravity, suction and pressure tanks
- Alternative sources, which are pump driven such as:
    - Private reservoir
    - Natural source such as rivers, lakes and underground water supply.

This report is concerned with the performance of these secondary water supply sources when attached alongside a primary supply and the effect that different methods of attaching these water supplies has on reliability of the system. In order to achieve this it is necessary to establish the reliability of each of these sources including that of the town's main. This can then be used to determine the reliability of the secondary water supply due to the variation in the class of the supply and in the physical components used in the primary and secondary supply. The primary focus of this report will remain on supplies within New Zealand supplemented with additional data from Australia, Canada and the United States of America.

1.2 Aim and objectives

Secondary water supplies have been included within the standard due to their perceived increase in the reliability of the water supplied to the sprinkler system, however, the decision on which secondary water supply best fits a building or region has not been adequately researched.

The aim of this report is to produce a general model of the reliability of the water supply system up to the start of the control valve enclosure. The control valves enclosure houses an assembly of stop and alarm valves, which typically mark the point prior to the supply entering the sprinkler system. The calculation of reliabilities at this point will include the variations in the different supply classes of the New Zealand standard. Including variations in the components enables an investigation into whether local effects generate a secondary supply class which is more reliable under a number of conditions.

The aim of this model is to produce an answer to the question; if the water supply is needed, what is the probability that water will be available to supply the sprinkler system?

1.3 Methodology

This section presents a methodology for the assessment of the reliability of the secondary water supplies under a single and dual supply source. The methodology considers the major elements that affect the reliability within the town's water supply as well as those from locally sourced water.

In order to perform an assessment of the water supply a number of questions need to be asked:

- What are the critical components?
- Do they relate to a fire engineering solution?
- If they relate to fire engineering solution, what is their reliability?

In many cases, components of the system are difficult to identify therefore it is important that a consistent level of uncertainty be maintained for these components throughout the report. In particular the components that make up the town's supply were difficult to categorise. In order to overcome this data was collected from a number of different sources and countries enabling the creation of a number of generic water transfer systems. As discussed, in the future work it would be possible to modify the data within the model for a specific supply if the components of the system are known.

Once values for the components were obtained, the Monte Carlo technique was used to combine the reliability of the individual system components into reliability value for the water system before it enters the sprinkler supply. By simulating random faults over a number of time steps the Monte Carlo technique allows for the combination of both components connected in series and parallel to be combined to obtain the overall availability of the water supply.

1.4 Scope

This report applies to secondary water supply systems installed in New Zealand and aims to remain within the context of the question stated in the aims and objectives by providing an answer to the question; if the supply is needed, what is the chance that it will supply the water required to the system? Under this context, the application is to secondary water supplies designed and installed in compliance with the recommendation in the New Zealand standard. The ability of a dual water supply to provide water will largely be governed by elements of the water supply that directly feed into the supply and this will be the focus of the report. However, large-scale natural events will also have an effect on the reliability of the system. For this reason, the report will include events such as droughts and earthquakes.

The design and reliability of the sprinklers systems attached to the dual water supplies are related to the occupancies of the building and can be design to comply with the standards in a number of ways. However, research into dual water supplies can be carried out independently of the sprinkler system installed. This allows for their exclusion from the scope of the report and connection downstream of the control valve assembly, this includes the sprinkler system and any related fire suppression equipment.

Therefore, the report will be limited to all components and factors from the source point through the distribution system and dual water supply up to the control valve assembly, which as discussed is typically the final point before the water enters the building's water supply and the suppression systems.

There is scope for further work to be conducted in this areas related to cost/benefits and to cover supplies outside the New Zealand Standard. However, this is outside the scope of this report.

1.5 Outline

The section summarises the chapters that appear in the report. The overall aim as stated above is construct a report for the reliability model of dual water installations. To achieve this, the report starts with an overall look at what are the dual water supplies available, the methods of distribution from the reservoir to the buildings, and the methods of analysis used to investigate the reliability. Before focusing on the data related to the specific assessment of the reliability of the water supply in different locations and the reliability of the secondary water supplies.

Chapter 2 Background

The process of water delivery and the infrastructure involved is important to the reliability of the end supply. Background in this area will involve the examination of the different delivery methods used within New Zealand as well as the details of the secondary supplies and the condition under which they are applied.

Chapter 3 Literature Review

This chapter provides a literature review of the topic. Covering discussion on what research has been performed in the area.

Chapter 4 Reliability of Water Supply Infrastructure

This chapter will present data collected from a number of sources to provide reliability figures for elements of the water supply such as pumps, pipe work, valves, tanks, filtration, and treatment. Secondary effects such as earthquakes and drought will also be examined to see their effect on the water supply.

Chapter 5 Analysis of the Reliability up to the Premises

Looking at a specific example system and comparing the results to those from other studies. In addition, the significance of each of the variables within the calculation is examined.

Chapter 6 Analysis and application of the secondary water supplies

This chapter will analyse the varying reliability that are achieved by applying the different available secondary supplies. The analysis provides for a comparison between the different types of secondary supply when compared against a single supply.

Chapter 7 Future research

There will be some additional work that can be continued after this report is completed. In particular, there will be the opportunity to add cost to the model, to assess the costs vs. benefit of the secondary water supplies.

Chapter 8 Conclusion

This chapter covers the conclusions that have been drawn from the research in this paper.

Chapter 2. Background

2.1 Standards

The New Zealand Standard NZS:4541 for sprinkler systems mandates the functional requirements of the system and the associated components required in the construction with the stated aim of ensuring that every sprinkler system has a reliable primary water supply.

The water supply of sprinkler systems under the standards is divided into three classifications aimed at providing supply reliabilities in decreasing order.

Class A. Dual superior supply. Dual supplies provided by a primary and secondary supply with only one of the supply sources dependent of the town’s main.

Class B Private Site fire main. Two supply sources, one of which is independent of the town’s main. Where a Class B supply differs from a Class A is by the inclusion of a private site reticulation system. This consists of a ring of pipe surrounding the property that can independently isolate sections of the supply network for repair and maintenance.

Class C Single supply. A single boosted water supply connection provided by a primary supply usually sourced from the town’s main. Used when the pressure of the water supply requires boosting. The single supply is divided into two sub-classes C1 and C2. The Class C2 superior supply increases reliability over the C1 supply by adding a second fire pump.

The standard requires that buildings presenting a greater risk, those building over 25m high or with a hazard factor (hazard factor represent the risk of an earthquake event and are tabulated based on location in NZS 1170.5) greater than 0.13 or where a compartment in the building is greater than 11,000m^2, be fitted with a dual water supply. If the hazard factor is below 0.13 a Class C2, supply is the minimum requirement under the standard.

2.2 Water Supply Class

As part of the aim of this report, it was stated that investigation would be conducted into the reliability of the different connection option. In order to achieve this the connection need to be broken down into their components. From section 602.2 and 602.3 of NZS 4541:2007 a number of combinations are available that can be used to supply the primary and secondary water supplies. These options provide the possible different combinations for connection of a primary and secondary supply and are applicable to both Class A and B2 supplies, while the primary supply is applicable to both types of a Class C supply. The options for supply connection are tabulated below.

Table 1 Supply options (specified in NZ 4541:2007)

Primary	Secondary
Town’s mains	Diesel pump supplied by other sources
Town’s mains	Electric pump supplied by other sources
Town’s mains	Elevated tank
Diesel pump supplied by other sources	Diesel pump supplied by other sources
Diesel pump supplied by other sources	Electric pump supplied by other sources
Diesel pump supplied by other sources	Elevated tank
Elevated tank	Elevated tank
Elevated tank	Electric pump supplied by other sources

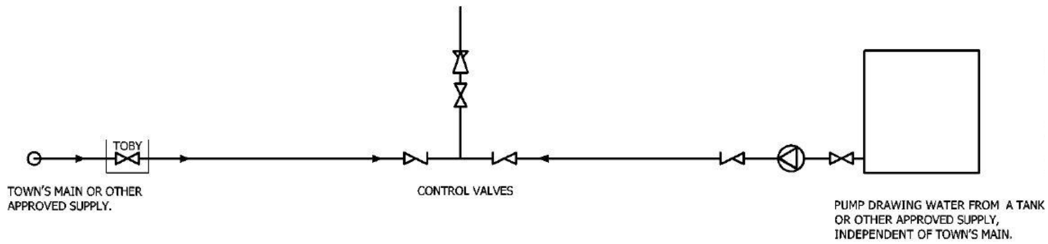
Other sources include: well, open water and tanks (on-ground or in-ground).

2.2.1 Class A Dual Superior Supply

The standard provides a typical layout of the installation of the applicable class of supply briefly described above. The figure below from the New Zealand

Standards illustrates what is expected in the installations for a typical Class A supply.

Figure 1 Typical Class A Supply from NZ 4541:2007



In order to model the reliability of such systems it is important to include as much detail as possible especially considering the overall similarity of the designs. For the illustration of a typical Class A supply, shown in Figure 1, a breakdown of all the components along with the possible supply connection (both primary and secondary) can be constructed.

Since the component required for a town's main connection are different to those required for locally sourced supplies two lists are required.

For a town's main providing the supply, the following components are required:

- Main's supply
- TOBY Valve, council controlled shut-off valve
- Pipe
- Check Valve

The components required when using a locally source water source as the primary and secondary supply are:

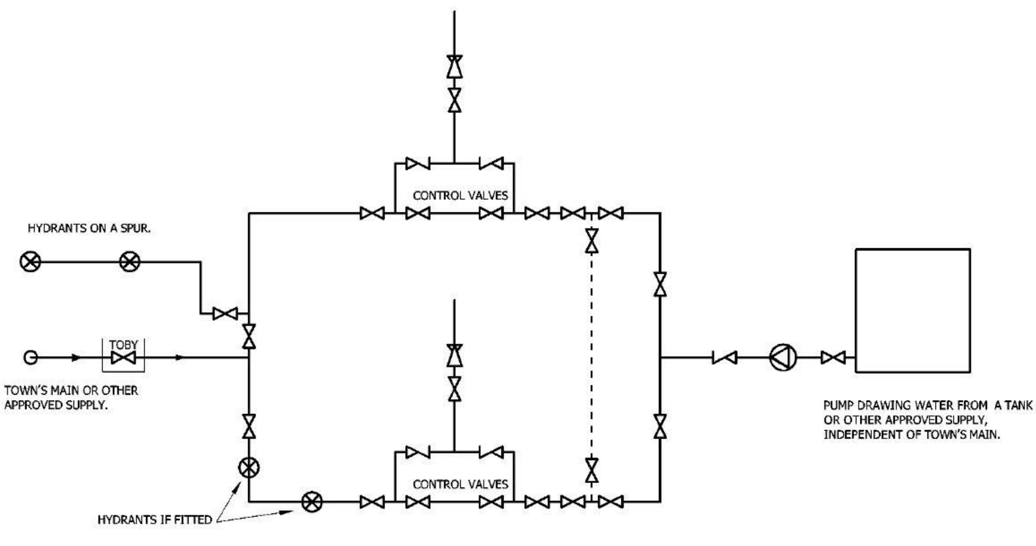
- Supply
  - Tank
    - Concrete tank with a roof
    - or Wooden tank with a roof
    - or Steel tanks with a roof

        - or Polyethylene tank meeting AS/NZ 4766 and secured
        - or Other provided design by chartered engineer
        - or Swimming pools meeting conditions outline in standards.
    - Or Wells and artesian bores
    - Or Open water
- Stop valve
- Pump
    - Diesel
    - Electric
- Pipe network

Class B Private Site Fire main

As with the Class A supply the standard provides a typical setup for a Class B supply. Class B supplies vary from the Class A supply by the inclusion of a loop main. This attempts the limit the pressure drop across the connections. However, this presents additional reliability issues due to the increase length of pipe and its possible exposure to earthquakes.

Figure 2 Typical Class B2 from NZ 4541:2007



As stated in the scope, this report is not directly concerned with reliability of the sprinkler system, only the water supply feeding it, this also extends to hydrant connections fitted of the site main.

The components required in the installation of a Class B supply.

Town's Mains

- Town's main supply
- TOBY valve
- Connected in parallel are
    - pipe

On-site Supplies

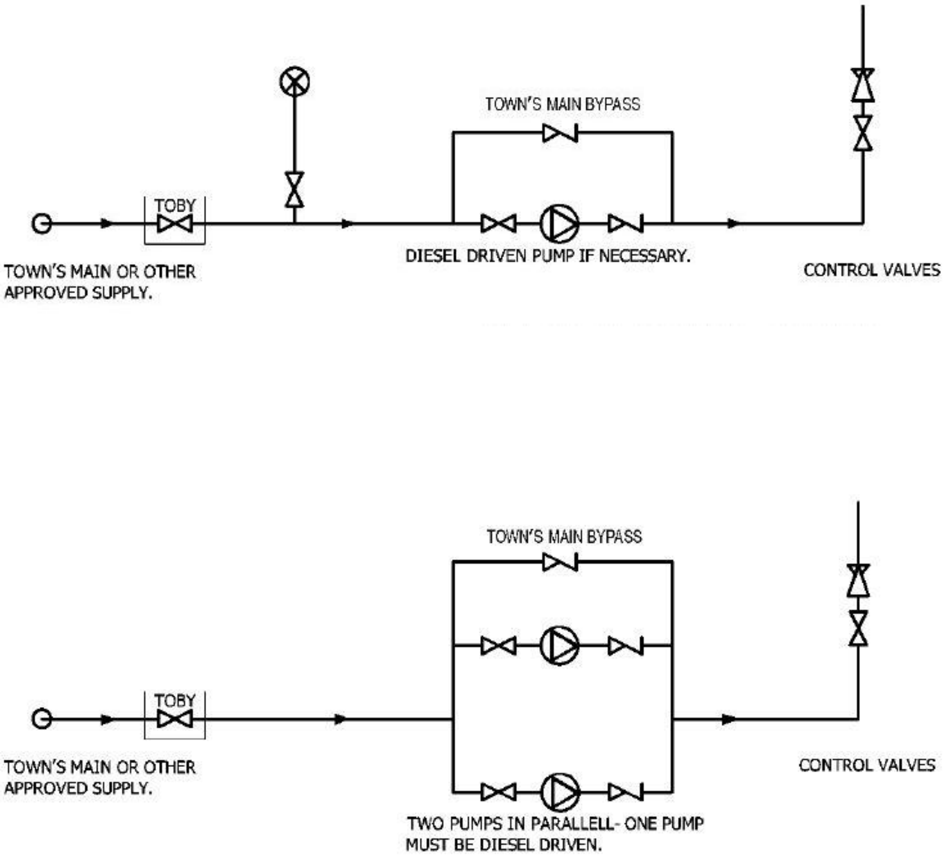
- Supply (as in Class A supply)
- Stop valve
- Pump
    - Diesel
    - Electric
- check valve
- pipe network

Class C

Class C supplies as mentioned previously are divided into two sub-classes both of which require only a primary supply. However, Class C2 supplies require the addition of a secondary pump aimed at providing greater reliability. This increased reliability is generally to the quality of the water supply maintaining pressure during a pump failure.

While the focus of the report is on the additional reliability of the secondary supply the Class C supply provides a datum from which to compare the additional reliability of the secondary supplies.

Figure 3 Typical Class C1 (top) and C2 (bottom) from NZ 4541:2007



2.3 Background to the water supply and area of reliability

Fundamental to the assessment of the reliability of a water supply system is an understanding of its design and operation. Typically, the operation of water supply systems can be broken into two distinct areas: headworks and distribution. The headworks covers the collections, treatment and transfer of water. The management of these areas is often separated to allow for the distribution of water from a single collection point to multiple demand locations.

This separation is one of the reasons that reliability assessments of water supplies are limited to one area of research.

2.3.1 Headworks

Headworks for this report shall comprise all of the following:

a) Supply source, i.e. bores, wells, dams, and weirs
b) Treatment plant
c) Pumping system, supply pumps from the supply source and booster pumps within the delivery system
d) Trunk mains, pipes from the point of supply to service infrastructure (treatment plants, pumps, storage tanks)
e) Tanks, online storage between supply sources and distribution

2.3.1.1 Collection

The principal water source for the majority of water supply is the natural run-off from catchment areas. This water enters the river, dams and lakes to create inflows. These inflows can exhibit large variations over time with the variability affecting the reliability of the water supply system as trade-offs are made to insure a constant supply.

These trade-offs can be managed in a variety of ways. However, in recent years the main method used has been Network Models. Within these models, a set of nodes is constructed and connected via links. Either these nodes can represent demand centres such as towns, agriculture, etc, or they can represent reservoirs. The links between these nodes has a cost of moving the water between the nodes. The model can then be used to determine the amount of water that can be supplied to the demand centres based on the inflow data collected over a number of years.

The application of these models in the water infrastructure management has produced a number of generalised models including WATHNET (Kuczera), REALM (Diment), and WASP (Kuczera). These models allow for the determination of the reliability of the bulk water being supply. This reliability excludes any effect the infrastructure has on the reliability and is most useful in fire engineering applications in determining the susceptibility of the reservoir network to drought or supply shortages.

Along with the reliability of the water demand, it is important to include the infrastructure, which is excluded from these models. This includes the reliability of elements in the collection system such as dam walls, weirs, pumps, and pipes used for bulk transfers.

2.3.1.2 Treatment and Transfer

Water treatment plants comprise a number of physical components to transfer, store and treat water, some of these components represent area of critical component failure and will affect the reliability of the system.
The treatment of the supply occurs in water treatment plants with foremost motivation of improving the clarity and to removing harmful organisms or pathogens in the supply. This treatment is conducted through a number of processing stages.

- Coagulation, flocculation and sedimentation form the first step in the process and are design to remove any particles from the water. The process involves the addition of chemicals to the supply to bind the particles together before sedimentation.

- Filtration involves the passing of the water through a sand or diatomaceous earth bed to remove any particles not collected by sedimentation

- The final treatment for the supply is the addition of a disinfection agent. The majority of the time this agent is chlorine and is used to kill the organism and pathogens in the supply.

In addition to these steps, chemicals may be added to the water supply such as fluoride.

The treatment plants do not represent a critical component in the reliability of fire system under short-term failures and commonly exhibit a high level of redundancy in the treatment of water in large urban areas.

The remaining water supply headworks system comprises of the transfer and storage of the water. The principal methods for these are pipe work, aqueducts, and tanks. These components generally represent critical components in the supply and their failure affects the reliability trade-offs associated with the modelling of the system.

2.3.2 Distribution

A water distribution system comprises of the physical components to distribute the water from the main pipeline supplies to the point of use. The distribution or reticulation systems are made up of a network of enclosed pipes, which can be supplement under additional demand by service reservoirs. The service reservoirs are typically enclosed tanks located in elevated position to maintain pressure within the system during peak demand. Service reservoir can also be located ahead of the water treatment plant depend on the topography of the system.

The pipe work within the reticulation system represents a critical component in the system and will have a direct affect of the reliability of the supply.

Chapter 3. Literature review

The operation of water supplies are typically divided into three distinct groups: the components under the direct control of the building owners, the distribution network, and the bulk water network. The operation of these three groups is often managed independently. For this reason, research into their reliability has been typically conducted in isolation.

Therefore, it is not surprising that a survey of literature and research contained very few references that addressed the performance of the entire water supply when attached to fire sprinklers.

Studies have looked into the reliability of secondary water supplies while conducting quantitative risk assessments into other topics. However, these stop at the distribution as the only source of fault in the system.

Thomas et al. (1992) conducted a risk assessment evaluating the reliability of fire safety system components. The research was commissioned by BHP's research division and was conducted during refurbishment of the office building at 140 Williams Street, Melbourne. The focus of the investigation was the effect that a fire would have on the steel structure of the building and the corresponding need to apply passive protection to steel structures. This research included estimates of the reliability of the town's mains and reliability of the components in the building's water delivery system, such as pumps.

Other studies by Bennetts at al. (1995,1998) were carried out as ongoing research by BHP into the effects of fire on steel framed buildings. These studies include further estimates of the town's mains reliability and the components associated with the fire suppression equipment.

Research conducted along a similar line to that of BHP's by Feeney (2001) into the performance of steel-based structures with sprinklers contained, an estimate of the town's mains reliability for Auckland based on data obtained from the water authority, Metrowater. Also included was a review of the reliability of components associated with sprinkler systems, and an estimate of the reliability of the secondary water supplies based on the previous New Zealand sprinkler standard (NZS 4541:1996).

Crawley (1993, 1995) described the use of HOMA, a water optimisation model, to assess the reliability of the Adelaide bulk transfer system. This research was aimed at optimising the reliability of Adelaide's water storage against pumping costs. Included in this research was a detailed review of the Adelaide bulk water transfer and storage systems, along with estimates of the reliability of each of the critical components. This aligns closely with the town's mains reliability being considered in this report, but is limited to Adelaide and does not included infrastructure outside the headworks system.

Crawley's model allows for the calculation of the length and number of periods the system would be without water based on water inflow, demand data, transfer costs, and restriction on use. Similar research conducted by Victorian University of Technology (Kuczera) and Monash University (Diment) have produced network modelling tools, WATHNET and REALM that perform similar optimisation calculations and are in much wider use than the model created by Crawley.

A number of governing authorities use these simulations to plan for the future uses and reliability of the water supply system. The South Australian Government (2007) and the Sydney Catchment Authority (2006) have produced reports that provide a number of water use scenarios and the reliability of the supply based on this usage.

Along with research into the systems as a whole, studies have been conducted into individual sections of the water supply chain. Donnelly (2006) looked at dam walls, Pim (1988) at tanks, and McElhanley (1996) at valves. These provide specific detail on the reliability of these components.

Water retailers provide information of the state of their networks in their annual reports. It is interesting to review this data, as the information supplied within these report can be used to determine the annual probability of failure for the reticulation systems and is comparable to the supply reliability used by Feeney (2001), Thomas et al. (1992) and Bennetts et al. (1995,1998).

The assessment of the reliability of the secondary water supply is to be conducted specifically on New Zealand based designs. Therefore, it is important to review the design requirements for secondary water supplies in New Zealand.

A review of the research into earthquakes was also conducted, as it is considered one of the main reasons for the division between Class A and Class B dual water supplies under the New Zealand sprinkler standard (NZ 4541:2007).

Adams (2008, 2004) discussed the effects that earthquakes had on dam walls and the susceptibility of different locations and construction methods. Ballantyne and Crouse (1997) studied the restoration of water supply after an earthquake and the probability of failure. Other research has been conducted into the specific probability of an earthquake causing damage to water supply infrastructure; a broader discussion on earthquakes appears in chapter 4.

Chapter 4. Reliability of Water Supply Infrastructure

This chapter of the report is concerned with the components that make up the off- and on-site water supply to the fire sprinklers. In this report, off-site components are those outside the boundary of the building and not under the control of the building's owner. On-site components are those within the boundaries of the building and controlled directly by the building's owner but, limited by the scope to those components before the control valve enclosure.

4.1 Reservoir Reliability

To gauge disruptions to the water supply by a failure of a reservoir, data was collected on the probability of dam failures indicating the length of time between failures. Data on the repairs of dams was not considered as it was assumed that repairs could be conducted without disruption to the supply of water.

Donnelly (2006) reviewed data collected from a number of sources on the failure rates of dams to determine the nature of the failures and the level of risk involved. The data collected showed that annual failure rate of dams from studies conducted in the United States, Japan and Spain were between $2x10^{-4}$ and $7x10^{-4}$ (failures/year), which equates to a mean time between failures of 1428 to 5000 years. The average mean time between failures indicated for these reports was 2500 years.

Also included in this review was the data provided to the Hydro Review. This review is the process whereby United States dam failures are reported. The data showed that up until 1999 there had been 421 dam failures in the United States with a mean time between failures of approximate 1600 years.

In comparison, the National Performance of Dams Program (NPDP) operated by Stanford University showed that, on average, the mean time between failures of dams from 1990 until 2007 was estimated at approximately 2250 years.

Clearly, the failure of a dam is an extremely rare occurrence. However, the widespread disruption and the lengthy construction time make these events worthy of consideration.

It is also interesting to compare the different dam construction methods to investigate whether one is superior to the others.

Donnelly (2006) also looked at the frequency of the failure for different types of dam construction. Based on the data collated in the FEMA/ICOLD (1979) , Federal Emergency Management Agency / International Commission on Large Dams, the report evaluated the frequency of failure as a percentage of the total number of dams built by that type. This showed that concrete buttress dam were the mostly like to have a failure a 2.6%, followed by; earth and rock filled at 1.2 %, concrete arch at 0.7% and the least likely to failure was concrete gravity dams at 0.3%. This data applies to the incidence of failure of dams reported in the United States.

Although, a large majority of the data is sourced from the United States it is expected that due to similar construction techniques and maintenance that the results for dams within New Zealand would be similar.

For this report, it is estimated that the reliability of a dam wall has a mean time to failure of 2500 years.

The construction period of large dams, those over 15 metres tall, covered in this report ranges between approximately 1 and 10 years from Crawley (1995) and SEQwater. The length of construction is dependent on the wall type and size. Risk and Reliability Associates (2009) estimated that the reconstruction of a reservoir in Gladstone, Australia, following a dam wall failure would require 1000 days.

For this report, it is estimated that the most dam walls could be repaired under ‘crisis’ conditions in 1000 days.

4.2 Pipe

During research into the headworks systems a number of key components were identified:

- Surge protection on the pipe
- On-line storage
- Energy dissipation valves
- Pipes

Surge protection is design to prevent damage to the pipeline during an unscheduled shutdown. If the surge protection fails or is taken out of service the system will still function.

On-line storage is provided to prevent a system failure under short-term outages. The online shortage can be by-passed in the event of a failure. However, this will reduce the supply’s resilience to other outages.

The energy dissipation valves are used to dissipate the hydraulic energy, a result of the difference in head on a closed section of pipe between the elevated storage and the distribution points. The design of these discharge valves allows for a number of redundant pathways to ensure that the supply remains unaffected in the event of a failure.

Crawly (1995) estimated that in the event of a major rupture of a section of the bulk transfer pipeline the system would be offline for a week and under “crisis” condition and that this could be repaired in one to two days. Based on the repair time the author felt the further investigation into pipe failures was not considered necessary in the examination of the bulk water transfer. Crawley's report was considering the transfer of water between dams and it was estimated that outages of less than seven days could be absorbed by the available storage.

Consideration must also be given to the pipelines connecting supply elements such as pumping stations and dams to the treatment plants. These pipelines have much less online storage available and even a short outage will have an effect on the supply reliability especially at peak demand.

On this basis, further investigation of these pipelines is the only element that needs consideration in the bulk water transfer system.

Price Waterhouse Coopers in an audit of the South East Queensland water asset found that breakages to main distribution pipes occurred at a rate of between 5.91/100km and 8.91/100km.

The data for pipe breakages covered in these audit inherently includes the physical and environment causes of pipe breakage. Al-Barqawi noted these environmental and physical factors when producing a model of the deterioration of pipes.

Physical causes included:

- Pipe type
- Diameter of the pipe
- Age and
- Breakage rate

Environmental factors included:

- Cathodic protection
- Ground water level
- Soil type surface type

- Road type

The age of the pipes and the pipe material had the highest contribution to the deterioration of the pipes.

For this report, it is estimated the breakages per km of pipe in the bulk transfer system is 0.07 (break/km) and the mean outage time for these breaks is assumed to be 90 minutes; estimated from the Nation Water Commission data that appears later in this chapter. The use of values from collected data was favoured of model data due to the complexity of the data required for the age, size and soil type (breakage rate) used to model a single water supply.

4.3 Pump

A number of components in the pumping stations were identified as critical components in the operation of the transfer system. These included the electrical side and the pumps themselves:

- Power feed
- Transformers and switchboard
- Pump motor and pump

The reliability of the power feed is determined later for a number of different locations and produced an availability of 99.965 %. (see section 4.6)

Estimates of the transformer and switchboard reliability were made for Adelaide’s bulk water transfer by Crawley (1995). These values are used in the report unchanged.

It was assumed that there would be little variation in the probability of failure of a large electric pump, used for bulk water transfer, versus a smaller electric pump used as a fire pump. The probability of failure of pumps appears to vary little across a range of sizes and applications from the data collected in the electric pump section (see section 4.7.1) of this report.

The on-line storage between pumping station and the distribution system allows for failures of less than a week to be disregarded. This boosts the reliability of the pumps as maintenance can be performed without a loss of supply. However, unlike the electric pumps used to boost the sprinkler system, the water supply pumps are run all the time at varying capacities.

Crawley (1995) estimated that the pump would have failure frequency of 1 in 20 years with a mean repair time of 7 days and the attached motor would have a failure frequency of 1 in 50 years and a 14 day repair time.

The failure frequency of the pump is consistent with that reported in Hydraulic Institute, which report the life span of pumps as between 15-20 years.

The availability of electric pumps and motors uses in this report for elements of the headworks was assumed to be 0.9998. This is calculated from the 7 days of repair time required every 20 years that was reported in Crawley.

4.4 Water treatment plants

As mentioned in the background, the main function of the water treatment plant is to clarify and decontaminate the water. A failure of either or both of these functions will have little short-term impact of the reliability of the water supply to the sprinkler system. Like many of the components in the headworks system, the water treatment plant can be by-passed to ensure continuation of supply.

Figure 25 in Appendix C shows the protocol for the treatment of water in the event that the water treatment plant fails. These measures are consistent with New Zealand water treatment protocol outlined by the Ministry of Health (2001). If water cannot be treated within the plant, consideration is typically given to issuing a notice for water to be boiled by consumers and not a shutdown of supply.

There are some circumstance in which the water supply will be completely turned off. These include when introduced chemical levels reach a pre-defined limit, however, these shut downs can be usually rectified within the time span allowed by the on-line storage. For these reasons, the impact on the failure of the water treatment plants is not included in this report.

4.5 Distribution (reticulation system)

To assess the potential for disruption to the town's main, data was collected from the Australian Government's National Water Commission (NWC), which includes data from most of the Australian water authorities but also contains some data from New Zealand. The data from the NWC covers the period from July 2002 until the last recorded report in June 2007. Listed in this data is the total number of properties connected to the authorities water supply, customer interruption frequency (per 1000 properties), water main breaks (per 100km), the length of the water mains, and the period of the interruption.

The interruptions recorded are defined by the NWC as a total loss of water supply due to any cause, both planned and unplanned. Unplanned outages covers shutdowns due to such causes as unscheduled repairs or ruptures; planned shutdowns cover periods of outages due to maintenance and repair of the network as well as outages due to new connections being installed and upgrades to existing pipe network.

Up until 2005-2006, reporting of all the values required to assess the reliability of the water supply was not commonplace and as of the end of reporting in 2009, some of the major water authorities still do not publish data on the duration of the shutdown. However, this is a small minority of the total number of suppliers and this appear to be changing with many of the councils and water authorities that are not collecting this data, reporting that it is being collected for the 2010 collection period.

As noted previously another source of data was from Feeney (2001), however, when Feeney collected this data for Auckland from Metrowater it was partially incomplete, as it did not have the planned outages recorded for the first twelve months. In a back-to-back comparison, it was shown that there was little variation ($1x\ 10^{-5}$) in the availability of the supply due to the inclusion of this incomplete data. As there is little difference in the values, the shorter period of complete data is used herein.

The table below shows a comparison of the figures obtained by Feeney for the Auckland supply, from Metrowater, compared to the results from the NCW for Brisbane's water supply.

Table 2 Summary of results for Brisbane Water from NWC and Metrowater (Auckland) from Feeney.

Location	Brisbane	Auckland
Period covered	June 2006 to July 2007	November 1997 to April 2000
Total number of water supply connections. (Property connections.)	435,000	115,000
Customer interruption frequency (note Auckland over 2.4 years)	144.7	513.32
Average length of interruption (minutes)	180.7	129
Expected shutdowns (minutes per year)	26	38
Frequency of shutdowns	4.98×10^{-5}	7.3×10^{-5}
Water supply reliability	99.995% (0.99995)	99.993% (0.99993)

The results from Auckland and Brisbane show the similarity in water distributions reliability but are also largely similar in shutdown length and customer interruption frequency, when the Auckland value is adjusted to a single year.

As the distribution (reticulation) plays such a significant role in the reliability, more analysis was conducted to ensure that no outliers or erroneous values were present in the data. The results of all the supplies with over fifty thousand connection in Australia are tabulated below; the second figure plots the histogram of these supplies.

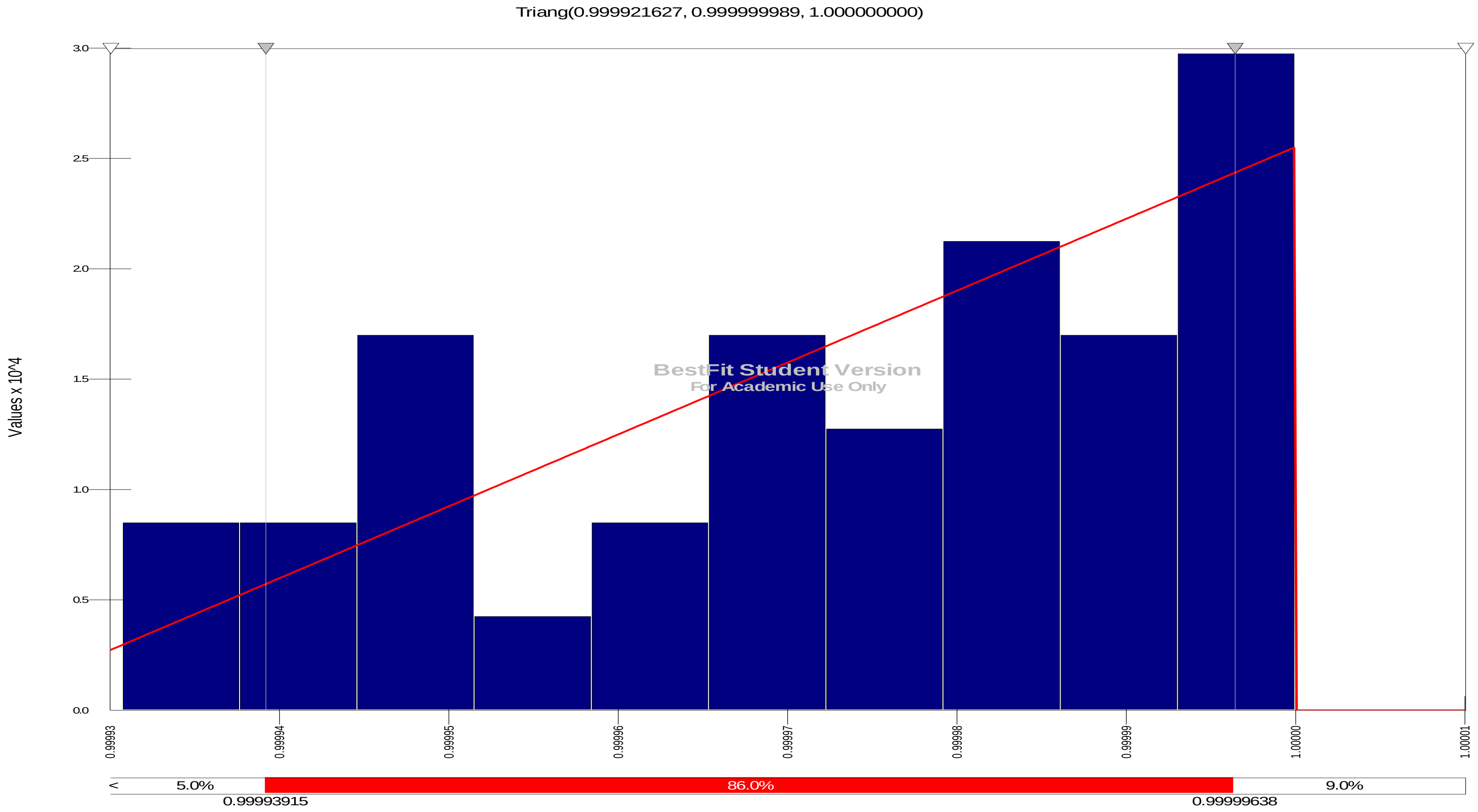
The remaining collected data for supplies under fifty thousand connections appears in Appendix E. These smaller cities exhibit a higher level of reliability than distribution networks found in the city with over fifty thousand connections. This is likely due to the smaller distances of pipes being easier to manage and a more disperse population resulting in less connections affected by a single failure in the distribution network.

Supplies with connection over fifth thousand was used as the default value for data from the NWC as it represents a slightly more conservative value than the average for all connection complied by the NWC.

Figure 4 Water Supplies with over 50,000 connections. Data from NWC (2007)

Water Supply - over 50,000 connections	**Total connected properties - water supply (000s)**	**Customer interruption frequency- water (per 1,000 properties)**	**Water main breaks (per 100 km of water main)**	**Length of water mains (km)**	**Average duration of an unplanned interruption- water (minutes)**	**Expected Shutdowns per year (minutes)**	**Frequency of Shutdowns per year (minutes)**	**Reliability**
Utility								
ACTEW	140	140.7	47.4	3,007	69.1	10	1.85E-05	0.999981
Barwon Water	127	214.8	-	3,431	126.7	27	5.19E-05	0.999948
Brisbane Water	435	144.7	49.7	6,340	180.7	26	4.99E-05	0.999950
City West Water	324	305.0	85.7	4,150	119.0	36	6.92E-05	0.999931
Hunter Water	217	370.0	37.4	4,638	176.4	65	1.25E-04	0.999875
South East Water	616	208.6	23.5	8,496	88.2	18	3.51E-05	0.999965
Sydney Water	1721	0.3	34.5	20,824	118.6	0	5.79E-08	1.000000
WC - Perth	680	63.5	13.1	12,527	145.0	9	1.76E-05	0.999982
Yarra Valley Water	651	303.9	57.3	9,018	84.3	26	4.89E-05	0.999951
Central Highlands Water	57	114.4	25.1	2,164	108.8	12.5	2.37E-05	0.999976
Coliban Water	65	80.0	42.1	2,115	112.5	9.0	1.72E-05	0.999983
Gippsland Water	59	163.0	36.6	2,001	93.7	15.3	2.91E-05	0.999971
Gosford	69	280.0	36.4	946	62.7	17.6	3.35E-05	0.999967
Goulburn Valley	51	180.1	3.9	1,677	121.0	21.9	4.16E-05	0.999958
Logan Water	66	37.9	11.0	1,257	39.6	1.5	2.86E-06	0.999997
Wyong	58	32.9	4.0	1,107	150.0	4.9	9.42E-06	0.999991

Figure 5 Histogram of the reticulation reliabilities from NCW (2007)



Values of note, from the data are the extremely small number of customer interruptions in Sydney and the low availability of the Hunter Water supply.

The values for the customer interruption frequency for Sydney are consistently low across a number of years. There are a number of factors, which affect the number of interruption experienced by a customer such as age of pipes, level of maintenance, amount of rain and soil type. Under the right condition, these can lead to the low level of interruption experienced in Sydney and since this is consistent over a number of years, this value is not considered an outlier.

The high level of disruption experience in the Hunter water supply is due to the high number of customer interruptions and the long duration in repairing of the ruptures and leaks. However, both of these values are consistent for a number of years and with the peak values reported for other regions.

There is a small possibility that an event within the distribution network would coincide with an outage elsewhere in the system. Since this value was assumed to be independent of the other outages there is a possibility that an event may be doubled up. For example, a pumping failure in the secondary water supply, failing at the same time as a pipe in the distribution network. Most of these duplicated events are accounted for when the failures are simulated in the Monte Carlo model and the small effect that these concurrent events produce will result in a more conservative result.

The NWC data and the values obtained by Feeney are the result of data collected over a period of one to three years. The result is an average annual value for the reliability of the water supply that does not take into account shorter period seasonal variations. Although this additional data would be useful in determining the variation in the reliability of the distribution network at a particular season or month, the average annual reliability obtained from the data remains consistent with the level of detail found for other elements of this report.

The analysis of the reliability of the water supply conducted here does not account for long-term variations in the reliability of the water distribution network. Some of these long-term variations such as drought can be accounted for by using the previously described network models, which take into account a much longer period of data collected from the inflow variation in a supply. There are effects that cannot be accounted for with other models or data, however, these effects typically result in small changes to the overall reliability.

The greater availability of reported data in Australia on water supply interruptions means that most of the analysis here is conducted outside of the New Zealand water supply network, with the exception of Auckland. It does not necessarily correlate that the values for Australia will apply to those in New Zealand. However, values for the water distribution reliability are consistent across a broad range of locations within Australia and those obtained for Auckland. This level of consistency is likely due to both New Zealand and Australia having the infrastructure and workforce in place to respond rapidly to disruptions in the water supply network and preventative maintenance in place to reduce occurrences of unplanned outages.

Bennetts et al. (1995) in their investigation into 140 William Street, Melbourne, also assessed the reliability of suburban Melbourne's water mains. It was reported that 90% of outages were repaired within 5 hours. This is consistent with the data obtained for the current length of repair time for Melbourne. The results of the reliability are tabulated below with a comparison of the current values.

Table 3 Comparison of Water Supply Availability

	1995 Figures (Bennetts et al.) Suburban Melbourne	2008-2009 (NWC) South-East Water, Melbourne	2008-2009 (NWC) City West Water, Melbourne
Frequency of shutdown per customer	5.8×10^{-5}	3.51×10^{-5}	6.92×10^{-5}
Expected shutdown per year per customer	31 minutes	18 minutes	36 minutes
Availability per year	99.994% (0.99994)	99.997% (0.99997)	99.993% (0.99993)

The results from the 1995 Bennetts et al. study compares well with more recently reported figures for suburban Melbourne water supplies. The values are also comparable to the previous discussed figures for Auckland (99.993%)

The average value for the cities of over fifty thousand connections was used.

4.6 Electricity

As part of the potential interruption to the water supplied from electric pumps, failure data was collected for the reliability of electrical supplies across Australia and New Zealand to indicate the frequency of the possible shutdowns of the electricity supply.

The frequency of shut downs is reported by the electricity suppliers (Orion Group (2009), Energex (2009), Gibbons (2008) and Ergon (2009)) as the System Average Interruption Duration Index, SAIDI. This represents the average total duration of the electricity supply interruptions, in minutes, experienced by an average customer due to unplanned and planned outages within a reporting period. Normally this is calculated on a yearly basis between the annual reporting periods. However, it can be calculated for other periods.

SAIDI can be used to calculate the availability of the power to the customer or ASAI (Average service availability) on an annual basis, by using the equation:

$$ASAI = [\frac{minutes\ in\ a\ year - SAIDI}{minutes\ in\ a\ year}]$$

Data collected from Western Power (Western Australia), Energex (South East Queensland), Ergon (South West Queensland) and Orion (Canterbury) has been analysed to show the availability of supply. The data collected relates to outage covering the period of July 2005 until June 2009.

The results show a clear difference between the reliability of power supply to CBD, urban and rural areas. This is not surprising given the increase in connection distance between customers in urban and rural area when compared to those in cities.

Table 4 Summary of Electricity Supply Availability

	2005/2006	2006/2007	2007/2008	2008/2009	Average
Availability % Perth CBD	99.998	99.994	99.989	99.991	99.993
Availability % WA Urban	99.96	99.95	99.95	99.94	99.95
Availability % WA Rural	99.91	99.89	99.89	99.89	99.89
Availability % Brisbane CBD	99.999	99.999	99.999	-	99.999
Availability % South East Queensland Urban	99.98	99.98	99.98	-	99.98
Availability % QLD Urban	99.95	99.97	99.96	-	99.96
Availability % Canterbury				99.98	99.98
				Average of all supplier	99.965

The United States Department of Agriculture (2009) suggested that a goal for rural utilities within the United States might be an availability of "four nines" or 99.99%. This is consistent with urban areas of Queensland (99.98%, 99.96%) and Western Australia (99.95%) but is an order of magnitude above that found in rural Western Australia (99.89%). However, the lower level of reliability in rural Western Australia is not surprising given the extremely low population density. The electrical reliability, like the water supply reliability, is not a measure of the quality of supply. It only pertains to sustained interruptions and not voltage fluctuation, abnormal waveforms or harmonic distortion. Interruptions of greater than 5 minutes are general considered a reliability issue, whilst interruptions of less than five minutes are considered a power quality issue. As a result of this, there will be a tendency to underestimate the total time of power outage and over estimate the reliability of the power supply.

The average reliability of the electricity supply was used, 0.99965, as it represents a more conservative result and covers the underestimation from quality issues. It is possible to the create a distribution from the data. This could then be used in the Monte Carlo simulation to place a value on the uncertainty in the model. However, to remain consistent with a majority of the data within the research this value was left as a discrete availability figure.

4.7 Components in the secondary supply

4.7.1 Electric pumps

In the assessment of 140 Williams Street, Melbourne, Thomas (1992) evaluated the probability of failure of the pumping components within the sprinkler's water supply. Included in this analysis, were the failures of an electric pump due to unavailability of mains power, faulty pressure switch to start the pump, and faulty electric pump.

Feeney (2001) summarised the results of this study and noted that the variations in the probability of failure of a Building Code of Australia designed building and a refurbished building were attributable to the differences in maintenance levels of the fire equipment. The Australian study was also modified by Feeney to reflect the stricter maintenance requirements in New Zealand that were in place at the time the reports were written. This result in a reliability for electric pump tabulated below:

Table 5 Adjusted Electric Pumps

	Probability of electric pump not working
BCA Building (Thomas et al)	7×10^{-2}
Refurbished Building (Thomas et al)	1.5×10^{-3}
New Zealand (Feeney)	2×10^{-3}

For comparison, results for electric fire pumps from Idaho National Engineering and Lees (2005) were used to estimate the probability of failure of an electric pump. Both these assessments relate to the nuclear industry; while not directly applicable to water supplies in New Zealand they do provide for a comparison of well-maintained pumping systems. However, unlike the Thomas and Feeney data these reports do not include factors outside faulty electrical pumps, such as electrical supply interruption and pressure switch failures.

An estimation of the electricity supply interruptions are provided above and an estimate of the switch's failure rate was obtained from Allen (1998) of 0.04552 failures per year. The result is an annual probability of failure of 2.14×10^{-4} for an electric fire pump installed with a nuclear power plant with "standard" electricity and switch reliability. This estimation is significantly more reliable when compared with the results the Feeney extrapolated from the Australian data. However, this is to be expected of systems installed in the nuclear industry.

For this report, Feeney's estimate of the annual probability of an electric pump was used, 99.8% or 0.998.

4.7.2 Diesel reliability

As noted previously, both Feeney (2001) and Thomas (1995) looked in to the reliability of pumps. It is interesting to compare reliability of the diesel pumps sourced from a number of different locations and industries to get a broader look at how various factors affect the reliability.

The first of these is the Offshore Reliability Data (OREDA) database that contains data collected by eight oil and gas companies in a wide range of components and systems used in the offshore platforms. The data collection started in 1984 and represents data from several geographical locations.

Along with the oil and gas industries, the nuclear industry provides detailed reliability data on a wide range of components and system installed in nuclear power plants across Europe and the United States.

The data is from the nuclear and oil and gas industries and whilst in the area of diesel fire pumps it will not be strictly comparable to the average New Zealand situation due to the differences environmental conditions and the levels of maintenance. However, they do provide a good point of reference to compare the values obtained by Thomas and amended by Feeney.

Table 6 Diesel Pump Reliability

Report	Annual probability of Failure		
OREDA 1995 1-100kw	7.4 x10^{-4}		
OREDA 1995 100-1000kw engine	4.8 x10^{-4}		
Ketron 1980 engine	3.31 x10^{-4}		
Idaho National Engineering lab (OREDA 1992)	3.33 x10^{-4}		
Thomas et al via Feeney	1.2x10^{-1} BCA 1.6x10^{-3} Refurbished 1.5x10^{-3} Feeney		
	BWR plant	**PWR plant 1**	**PWR plant 2**
Rowekamp and Berg 2000 (stationary fire pump) nuclear	2.1x10^{-6}	1.4x10^{-6}	3.8x10^{-6}

The values from OREDA, Ketron, and Idaho National Engineering laboratory were reported as a mean time between repairs. This needs to be transformed into an annual probability of failure for comparison to the results from Feeney.

The oil and gas industry results reported in OREDA are in the lower range of reliability and are close to those reported by Feeney for a BCA building. As would be expected of a high-risk environment, the nuclear power plant’s fire pump reliability falls in a range offering higher reliability. The variation in the values reported in nuclear power plants is indicative of pumps that would be found in building applications, with maintenance playing a large role in the reliability of a diesel pump.

The value for the annual probability of failure of a diesel pump was taken as 1.6 x 10^{-3}. This represent the probability a diesel motor will not start or will failure during it operation and was taken from Thomas via Feeney.

4.7.3 Valves

The New Zealand standard specifies the valves that must be connected to the primary and secondary water supplies. The table below shows the expected failure rate of these valves.

Table 7 Valve Reliability from McElhanley (1996)

Type	**Failure rate (failures/ year)**
Check	0.0133
Stop Valve	0.002

4.7.4 Tanks

Elevated water tanks make up one of the three alternatives for a primary supply source, however, tanks may also be installed in a non-elevated position when a pump is attached. To gauge the availability of water supplied from these sources, the construction material used, the location (Elevated, in-ground or on- ground) of the tank, and the amount of time that the tanks is empty have been analysed to indicate the annual probability of failure of tanks.

Australian and New Zealand Standard, AS/NZ 4766:2006, covers the construction of polyethylene storage tanks and requires the tanks be constructed to have a design life of 10 years. However, the Standard does not cover the installation of tanks underground. To append the Standard, Water Services Association of Australia (WSAA) have created a report, WSA 128, with the objective of providing design, manufacturing and performance for buried rainwater tanks, where recommendations are not available under the Australian and New Zealand Standards. Recommendations are provided for two different criteria:

- Tanks installed clear of any building and away from structural element –a design life of 25 years is recommended
- Tanks installed under or close to building where structural elements of the building may be affected – a design life of 50 years is recommended

The WSAA recommendations cover the installation of glass fibre, plastic and concrete tanks.

In comparison to the of the recommendation by the WSAA, De Walle (1981) found that for concrete septic tanks the failure rate was between 1 and 5% per year. If, the failure rate of the tanks is treated as an exponential curve. A failure rate of between 1% and 5% at the end of the first year yields a mean time to failure of 99.45 years and 19.5 years. However, this excluded the effects of a burn-in period were a large number of failures are likely to occur during commissioning.

The results from DeWalle (1981) are confirmed in a study conducted by the Philadelphia Suburban Water Company into the life cycle costing of concrete vs. steel tanks, which showed concrete tanks in service with the Philadelphia suburban Water Company had service life of 48 years with repairs conducted 20 years into service. The results also showed that the steel tanks had a service life of over 45 years with repairs conducted 17 years into operation. The main determination to end the use concrete tank was cost of the repairs, and operation and not the failure of the tank.

The data from these studies falls into a range above the typical warranties for commercially available tanks with plastic tanks having a typically warranty period of around the 25 years, coated steel 20 years and concrete 15 years.

Mean time to failure was estimated at 50 years for non-elevated and 25 year for elevated. The repair time of elevated and on-ground tanks was estimated at 1 day under “crisis” and 3 days for underground tanks.

4.8 Weather and Natural disasters events

The data collected so far does not include long-term natural events such as earthquakes. The incidences of these natural events are typically rare and are not always of a magnitude to result in damage to the water system. However, the consequence of these rare events can be severe and are not statistically independent. These events may produce damage to almost every component of the water supply system (water tanks, town’s main, pumping station, etc.) and some of these events may result in a fire occurring.

When looking at the effect that natural disasters have on the supply system a number of events were considered:

- Icing or snow damage to pipes or supply source;
- Contamination (eg. algal outbreaks) that cannot be controlled by treatment plant;
- Earthquakes;
- Droughts.

Damage from snow or ice and contamination to the supply was disregarded due to the limited effect on the supply. Mitigation allows for the intakes to be moved below contamination or for the contamination to be treated. Similarly, icing of rivers and lakes can be mitigated with design.

Although the occurrence of earthquakes are rare, the propensity is for damage to be spread over a wide range of the water supply infrastructure. This coupled with the threat of fire caused by the earthquake leads to its detailed inclusion in the review.

The exhaustion of water during droughts is as much about the operation of the water supply as it is about the weather around the catchment areas. For this reason, water authorities typically conduct extensive modelling on the reliability of the water storage, evaluating the level of storage against usage trade-offs. This coupled with non-independence of secondary supply such as tanks, rivers, and lakes means that it is appropriate to include a detailed review of droughts.

4.8.1 Earthquakes

The New Zealand sprinkler standard recognises the increased threat from the ground motion during earthquakes and requires that building in earthquake prone areas, a hazard zone greater than 0.13 and taller than 25m, have a Class A water supply. This includes two independent supplies only one of which is reliant on the town's main and reduces the amount of ground pipe work install around the building.

However, earthquake damage does not just affect the pipe network around buildings. A large portion of the headworks is susceptible to earthquake damage and must be included into this research.

The likelihood of a loss of integrity of a reservoir varies depends on the method of construction, the age, and the quality of the workmanship. Lin and Adams

(2008) noted that of the 133 dams in the Canadian hydroelectric scheme that earth/rock filled dams were considered more vulnerable to earthquakes than concrete dams. This is backed-up by the ATC (1995) which applies a damage state to each construction method based on the MMI (Modified Mercalli Intensities), a measure of earthquake intensity.

Since the dams considered in Lin and Adams (2008) study were constructed between 1910 and 1996, it was considered necessary to account for the age of the dam in their vulnerability to earthquakes. Older dams built before 1930 having been constructed with no seismic consideration and those constructed between 1953 and 1985 having been constructed to a much lower seismic hazard than current requirements. A differentiation was also made between the seismic regions of Canada with dams in Western and Eastern Canada having different exposures. The result was that dams constructed pre-1930 out of earth-fill in Western Canada (highest risk) had an annual probability of sustaining slight damage of 0.00126. While, an earth-fill constructed in Eastern Canada in the 1950's (ranked 17th) had an annual probability of slight damage of 0.00088.

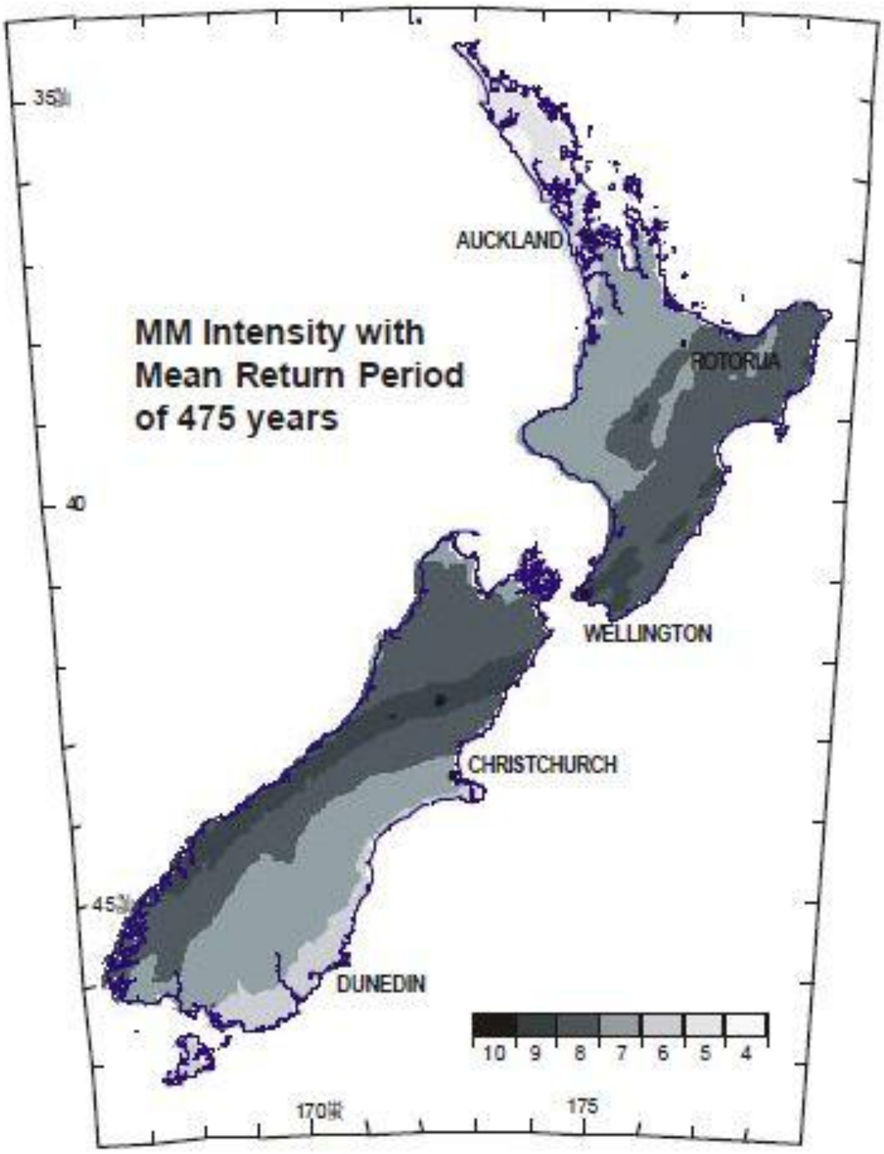
Ballantyne and Crouse (1997) noted that following the Loma Prieta and Northridge earthquakes in California and the Kobe earthquake in Japan there was major loss of water supply. In these earthquakes, the pipelines in both distribution and transmission of water failed and the reservoirs drained.

It was also shown that historically the major cause of system failure relates to the damage of the bulk water transfer and the distribution pipe network. Other failures occurred in the treatment plants, pumping stations, tanks, power systems, and building structure. A loss of power was identified as the greatest failure mechanism. However, it was found that the water treatment plants and pumping station failures contributed only as secondary cause of the system failure.

The report states that anchored tanks generally perform well during earthquakes, provided they are design transfer force from the shell into the foundations. Unanchored tanks fair less well with a number of large tank failures.

Ballantyne and Crouse presented a risk assessment of the failure of a water system in California. An adaptation of the fault tree presented in their report along with the values for the failures appears in Appendix C. The fault trees have been modified for the supply condition present in this report with the omission of treatment plant failures relating to water quality and the modifications to allow for the infrastructure consistent with those present here.

Figure 6 MMI Intensity with Mean Return of 475 years for New Zealand from Falconer.



Whilst the research was conducted into areas of California and Canada, the infrastructure is similar to that that appears in New Zealand. The magnitude and intensity of earthquakes in California and Canada will differ from those in the New Zealand. However, this is also true of earthquakes within different regions of New Zealand as illustrated by the Figure 6, which shows the MMI for a 475-year return period.

As a consequence of the seismic activities, pipes within the town’s distribution system and those within a properties’ supply can be damaged, limiting the availability of water. While Ballantyne and Crouse recognised this threat and provided an estimate of the reliability of the distribution and bulk transfer, the site main required a failure rate based on the length of pipe installed. PAHO (1998) recognised this need and used the 1991 Limón earthquake in Costa Rica to estimate the expected number of breaks in pipelines affected by seismic activity.

PAHO studied the vulnerability of pipes under a number of conditions and expressed their failure rate in terms of breaks per kilometre. The factors that affected the pipes failure rate included the profile of the soil, the potential for liquefaction, and the intensity of the earthquake these factors resulted in the calculation a seismic hazard factor for a location.

Table 8 Soil Profile – taken from PAHO (1998)

Soil Profile	Description	Hazard
Rocky	Rocky Strata, or soil with wave propagation greater than 750 m/s	1.0
Hard	Well consolidated or soft soil to depth of less than 5 metres	1.5
Soft	Soft soil in excess of 10 metres	2.0

Table 9 Liquefaction Hazard– taken from PAHO (1998)

Liquefaction	Description	Hazard
Low	High drainage, low sand, and well consolidated soil.	1.0
Moderate	Moderate draining soil with moderate sand content.	1.5
High	Poor drainage, high water table, and high sand content.	2.0

Table 10 Permanent Displacement Hazard– taken from PAHO (1998)

Displacement	Description	Hazard
Low	Low slope, well compacted fill not near river or faults	1.0
Moderate	Slope less than 25% compacted fill close to river or fault	1.5
High	Slope greater than 25% near a river or fault with uncompacted fill.	2.0

The values are multiplied together to produce the seismic hazard factor. PAHO suggested that a value of less than 2 for the seismic hazard factor represents a low seismic hazard, values between two and four a moderate seismic hazard, and values equal to or greater than 4 a high seismic hazard.

This seismic hazard can be coupled with the Modified Mercalli Intensity, shown in **Error! Reference source not found.** for New Zealand, to produce the xpected number of failures per kilometre of pipe. The number of breaks caused by an earthquake in cast iron pipes for the different Modified Mercalli Intensity is given in Table 11.

Table 11 Pipe fault per km based on Hazard– taken from PAHO (1998)

Mercalli intensity	Faults per kilometre	
	Seismic Hazard Factor < 2	Seismic Hazard Factor > 2
VI	0.0015	0.01
VII	0.015	0.09
VIII	0.15	0.55
IX	0.35	4.00
X	0.75	30.0

The values can be modified away from cast iron to other material using the conversion table shown below.

Table 12 Conversion for Pipe Material– taken from PAHO (1998)

Material	Correction factor
Steel	0.25
Cast Iron	1.00
PVC	1.50
Asbestos Cement	2.60
Reinforced concrete	2.60

Modification to the breakages per kilometre to allow for the age of the pipes can add up to an additional 50% for old/poor condition pipes. As narrower pipes are more susceptible to breakage, an additional 50% can be added for sub 75mm pipe and 25% for pipes between 75mm and 200mm.

O'Rourke and Bouabid (1996) applied a similar approach to concrete pipe damage in Mexico City due to the 1985 Michoacán event, an earthquake with an MMI of between VIII and IX. This produced an estimate of the damage ratio of 0.3 to 0.5 repairs per kilometre. This compares to values calculated using PAHO method of 0.39 to 0.91 breaks per kilometre for a low seismic hazard. The greater range is large due to the discrete steps in the earthquake intensity used by PAHO.

Shinozuka asserted that the occurrence of these pipe breaks followed a Poisson distribution. It was also stated that there are four states of pipe breakage; no breaks (no damage), one or two breaks (minor damage), three to five breaks (moderate damage) and equal to or greater than six breaks (major damage).

A Monte Carlo simulation based on the probability of the pipe having minor damage through to major damage was coupled with the breakage per km to evaluate the damage caused by earthquakes to the pipes.

Given the variability in the inputs for the magnitude of the earthquakes, the locations, and the infrastructure involved, the probabilistic assessments here can only give an estimate of the magnitude of the effect earthquakes have on the water supply. However, attempts have been made to cover the highest proportion of this variability by covering the soil condition with the model of the pipe breakages.

The seismic forces covered under the New Zealand Code of Practice for General Design and Design Loadings for Buildings NZS 4203:1992 includes an accepted exceedance factor of 0.1 in the 50 year design life. This exceedance probability yields a similar return period to the values used in Figure 6 and represents an annual exceedance probability of 0.0021 for seismic and gravity forces.

4.8.2 Drought

The statistics collated for the distribution and storage of water so far do not cover the longer-term effects of weather on the system. Although, the occurrence of drought in New Zealand is relatively rare occurrence, variability in weather patterns can even affect areas with relativity high rainfall. The capacity of the storage is related to the climatic condition as well as the demand placed, by the consumer, on the water supply system. This demand will vary with the climatic conditions, with these variations also affecting the inflow into the reservoirs. During hot dry periods, demand would be expected to be high and inflow would be expected to be low. For this reason, it is beneficial to have a close management of the supply.

The strategic management of these supplies has also changed in recent times in part due to concerns about the public health of drinking water, the decline in the reservoir location, and the environmental impact of construction of new dams and reservoirs. This transition has lead water authorities to focus on achieving the maximum efficient from the available water supplies.

As a consequence of the closer management of the water supplies, service standards are being designed to ensure supply continuity and to limit the duration and frequency of water restrictions and shortages. With the aid of network modelling programmes such as WATHNET and REALM the resilience of the water supply to drought can be assessed and adjusted on a daily basis.
Thus the ability for a supply network to resist drought is as much a result of the cost of suppling the water and the restrictions placed on it usage as it is the amount of inflow into the supply. Therefore, the reliability of the systems is often the result of a yield being placed on a system to account for population growth, cost, and drought and still meet the service standards. The infrastructure and system design are typically modified around this yield to ensure supply.

A review of the Sydney water system in 2007 estimated that for a snapshot of the yield status in December 2006, that the limiting factor in supply was the security yield. The security yield was defined as maximum amount of water that could be extracted out of the storage system such that reservoirs did not approach empty (less than 5% capacity) more than 0.001% of the time.

In comparison, a review of the assets in Greater Wellington's water supply showed, that for the current supply targets, that sufficient water would be available on a daily basis to meet the 1 in 50 year return period of a drought situation with additional hosing restriction in place.

The probability of the water supply being unable to extract water is less than the 0.001% expressed in the Sydney review as the yield available is dynamic and as water supplies approach exhaustion restriction applied on its use become more severe. However, the Wellington supply is only set to meet a drought with a 1 in 50 year return period. This offers only an insight in to reliability over a much short period than that of the Sydney supply.

It is assumed that the probability of the water supply being exhausted of water because of drought is 1×10^{-4}. This roughly equates to a loss of supply for three and half days ever one hundred years. This is only an estimate and aims to cover a wide range of supply sources under the same level of accuracy provided for other components in the system. For a specific location, it would be possible to extract the daily data from one of the network models to produce a more site-specific result.

Chapter 5. Analysis up to the premises

5.1 Adelaide Case Study

It is important to verify that the values obtained in chapter 4 can be used to generate data that reflects the interruptions in a real water supply. In order to verify these potential interruptions, a comparison was made to the values obtained by Crawley (1995) for the Adelaide water supply.

The data compared covers the northern metropolitan water supply and storage system of Adelaide, which is comprised of three sub-groups: the South Para subsystem, the Little Para subsystem and the Torrens subsystem.
Figure 7 shows the area supplied by:

- South Para/ Little Para subsystems
    - Barossa and Little Para water treatment plants
- Millbrook and Mannum-Adelaide pipeline (Torrens System)
    - Anstey Hill treatment plant
- Hope Valley Reservoir (Torrens system)
    - Hope Valley treatment plants

The supply and storage of water provided by these subsystems are very different. The Torrens systems supply of water comes principally from the Murray River through a series of pumping stations. While, the South Para subsystem is a series of reservoirs that are interconnected through rivers and pipes. During the summer months, the South Para reservoir is supplemented by water collected in the Little Para subsystems, which functions largely as balancing storage.

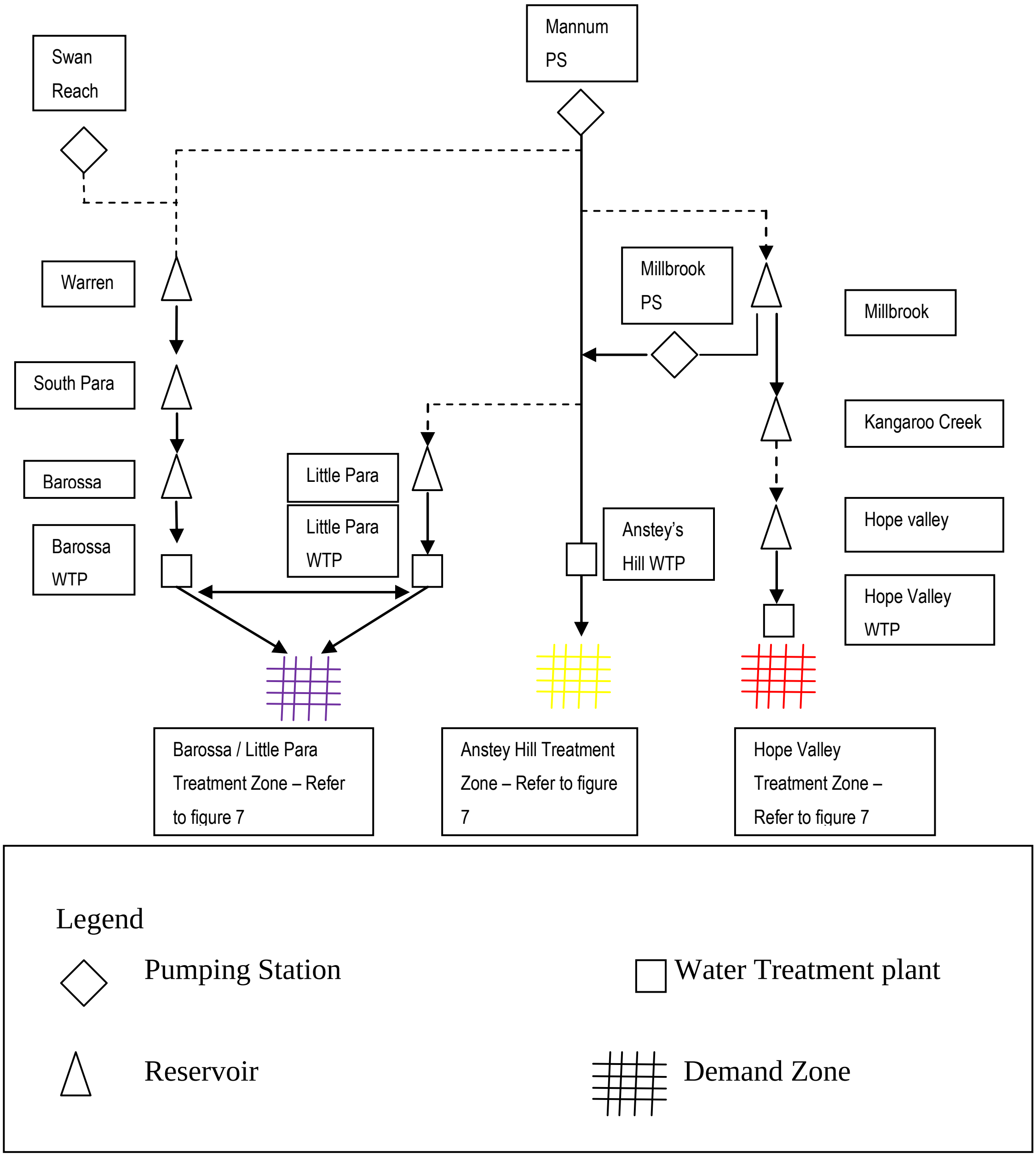
The water supply from these water treatment plants is assumed to supply only the areas marked. In reality, the boundaries of the treatment zones would be less defined with these boundaries overlapping. However, it is more conservative to assume that only a single supply is feeding water into these areas. This is

considered realistic for most areas in the supply.

Figure 7 Water Treatment Zone for Adelaide defined by South Australian Government's online mapping, South Australia Government (2010).

The three subsystems and four water treatment plants that comprise the northern metropolitan water supply and the four demand centres, which they serve, are shown in the schematic in Figure 8. While not shown on the schematic, there are some much smaller online demand centres, which are fed from the upper reaches of the dam and pumping systems.

Figure 8 Schematic of the Northern Adelaide Demand Centres



At this stage, the components used to balance the levels of the dams can be disregarded (shown in Figure 8 with dashes). The reliability of these bulk transfer lines falls under calculations made in the optimisation of the system, and their reliability is included in the probability of the dam holding sufficient water to meet demand, which is discussed in the drought section of this report.

5.1.1 South Para Subsystem

The South Para branch of the Adelaide system is made up of three reservoirs:

- Warren Reservoir
- South Para Reservoir
- Barossa Reservoir

The Warren Reservoir was constructed during the years 1914 to 1916 on the South Para River to provide storage and supply to the Barossa Valley. The reservoir is supplemented by water pumped in via the Swan Reach-Stockwell (2020 ML/month) and Mannum Adelaide Pipelines (420 ML/month). The dam wall is a concrete gravity construction and the reservoir holds 4.77 GL. The Warren Reservoir is connected through spillways and rivers to the South Para Reservoir. The reservoir and spillways are design such that the Warren Reservoir will spill over into the much larger South Para Reservoir in the late winter months of most years.

The South Para Reservoir was constructed during the years 1948 to 1958 on the confluence of the Malcolm and Victoria Creeks. The dam wall is a rolled fill dam wall, which holds approximately ten times the capacity of the Warren Reservoir at 44.8GL. The reservoir is situated 350 metres above the demand centre in Adelaide and is connected to the South Para Reservoir by a diversion tunnel and aqueduct off the Barossa Weir, the Barossa Reservoir is the final storage point in the South Para system.

The Barossa Reservoir was constructed between 1899 and 1902 the reservoir is separated from local catchment to improve water quality. The reservoir was not linked into the northern metropolitan supply until 1940. When the first of two trunk lines were built. The reservoir holds 4.51 GL behind a concrete arch wall and is situated 250 metres above Adelaide, which allows water to be gravity fed through the filtration plant to the demand centres.

The Barossa Reservoir feeds into the Barossa water treatment plant which has a capacity of 160 ML/day. This matches the supply available through the trunk mains.

5.1.2 Little Para Subsystem

The Little Para Reservoir is the most recent addition to the Adelaide supply. It is located on the Little Para River at an altitude of approximately 200m above the demand centres in metropolitan Adelaide. It was commissioned in 1979, with a dam wall constructed of concrete faced rock and a capacity of 20.8GL. The Little Para reservoir is used as a ground water recharge with supplemental water supplied from a branch off the Mannum-Adelaide pipeline. The primary function of the Little Para Reservoir, however, is to act as a balancing storage (this enables water to be retained during winter, when demand is less than supply, and utilised at peak periods during summer) for the South Para Reservoir.

Water treatment is conducted at the Little Para treatment plant, which was constructed in 1984. The plant has an identical supply rate as that of the South Para Treatment Plant of 160 ML/day.

5.1.2.1 Component reliability

For the metropolitan Adelaide supply the South and Little Para subsystems join to supply the Barossa and Little Para region. The critical components in the transfer and distribution of the bulk water for the South and Little Para Subsystems have been identified and estimates of their components' reliability are presented in Table 13.

Given the storage capacity of the Barossa Reservoir, it is possible to disregard the effect of a failure in the underground diversion and aqueduct, which connect the South Para Reservoir to the Barossa Reservoir. The underground diversion and aqueduct operating in parallel almost assures that any failure within these components can be repaired before the supply within the Barossa Reservoir would be depleted.

Table 13 Components in the South/Little Para Systems – Summary of values obtained in Chapter 4

Component	Mean Time to fail (days)	Frequency (1/years)	Daily Availability
River			0.99990
Warren Reservoir	1000	2500	0.99890
South Para Reservoir	1000	2500	0.99890
Barossa Reservoir	1000	2500	0.99890
Trunk Main 1, based on length	90 (minutes)	0.21	0.99996
Trunk Main 2, based on length	90 (minutes)	0.21	0.99996
Distribution	from data		0.99995
Little Para Reservoir	1000	2500	0.99890

The remaining components can be combined to produce a reliability figure for the Barossa/Little Para treatment area using a Monte Carlo simulation to generate synthetic failure data for the metropolitan Adelaide's bulk water system. These randomly generated failure events, represent the transition between states (working and failed) of all the components in the South/Little Para subsystem over a nominated number of steps. The results for the South Para and Little Para supply are presented in Table 14.

Table 14 Availability of the Headworks

	South Para	Little Para	Combined
Availability	98.927% (0.98927)	99.233% (0.99233)	99.992% (0.99992)

There are no interim states of operation for the supply. This is due to the equal capacity of both the Barossa and Little Para water treatment plants and the redundancy offered by the Little Para system. Therefore, the supply of water to the treatment area is either; working or failed.

The design of the system with separate water treatment plants only requires the addition of the reticulation data to calculate the availability of the supply at the boundary of a property. With the data for Adelaide's reticulation not available, the average for Australian cities with over 100,000 connections was substituted. This produces the availability shown in Table 15.

Table 15 Availability of the Water Supply in the Barossa - Little Para Treatment region

Availability of supply to a premise within the Barossa/Little Para treatment area	99.987%

Further discussion of the data is provided later in this chapter, however, at this point it is noted that there is some additional reliability provided by the simpler layout of the Little Para system and as would be expected, the parallel connection provides a higher reliability state than the single supply.

Crawley (1995) did not provide a calculation for the reliability of the reservoir systems. In the report, it was assumed that the failure of these reservoirs was not significant and lay outside the work of optimising the water supply.

5.1.3 Mannum-Adelaide Pipeline

The Mannum-Adelaide pumping system was constructed as a means of supplying water from the Murray River to metropolitan Adelaide. Barrages were constructed in 1935 at the mouth of the Murray River to prevent seawater entering the mouth of the Murray River. Once completed, work began on the 60 km of pipeline which connections the river township of Mannum to the terminal storage in the suburb of Modbury. The pipeline consists of three pumping stations used to elevate the water over the Mt Lofty Ranges and into Adelaide's supply. A longitudinal schematic of the pipeline, storage, and pumping station is shown in Figure 11.

The components of all three of these pumping stations are similar in nature, allowing the transfer of pumps and electrical components to other pumping stations in the case of a failure. The addition of an inlet and four vertical pumps to extracted the water from the Murray River, shown in Figure 9, are the only difference between pumping station 1 and the other two pumping stations, (pumping station 2 and 3), shown in Figure 10.

Figure 9 Pumping Station 1

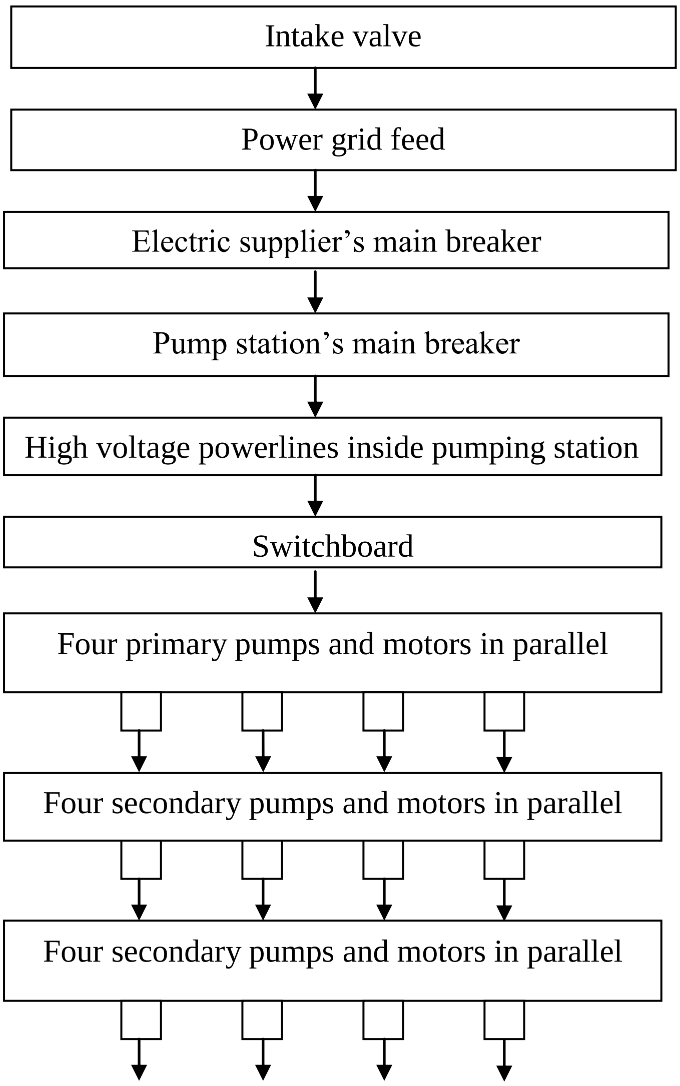
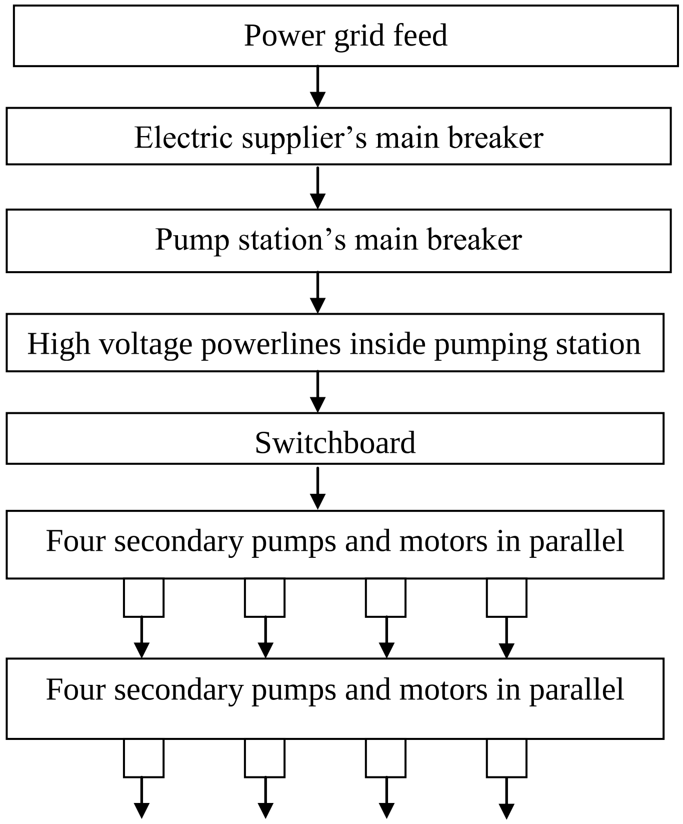


Figure 10 Pumping Stations 2 and 3



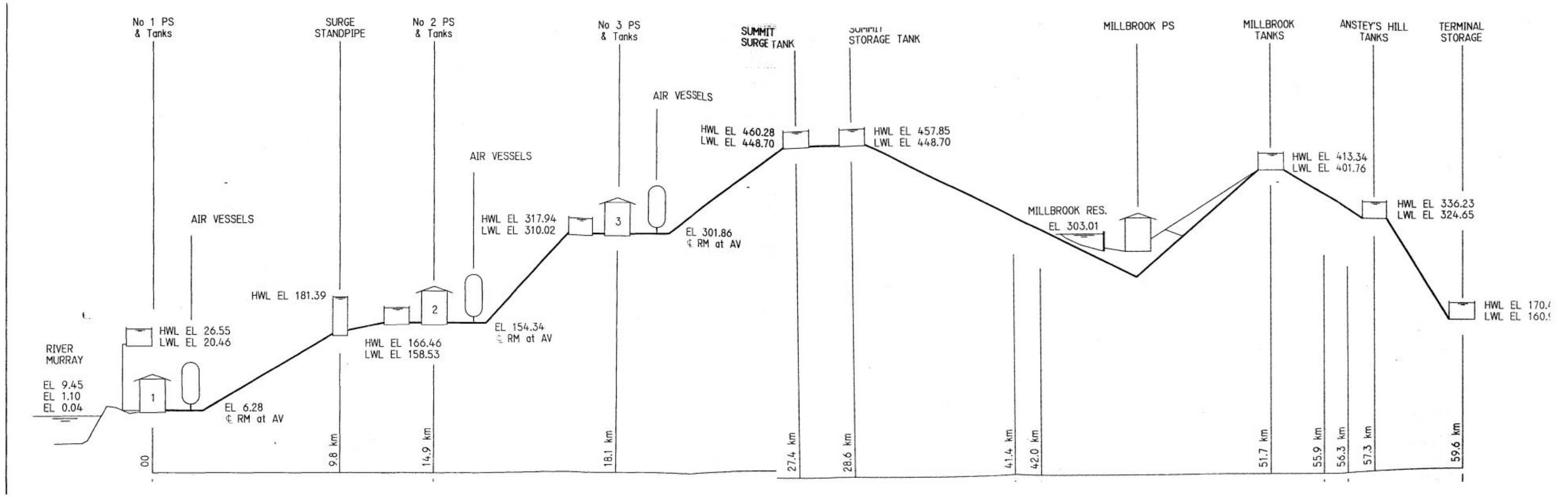


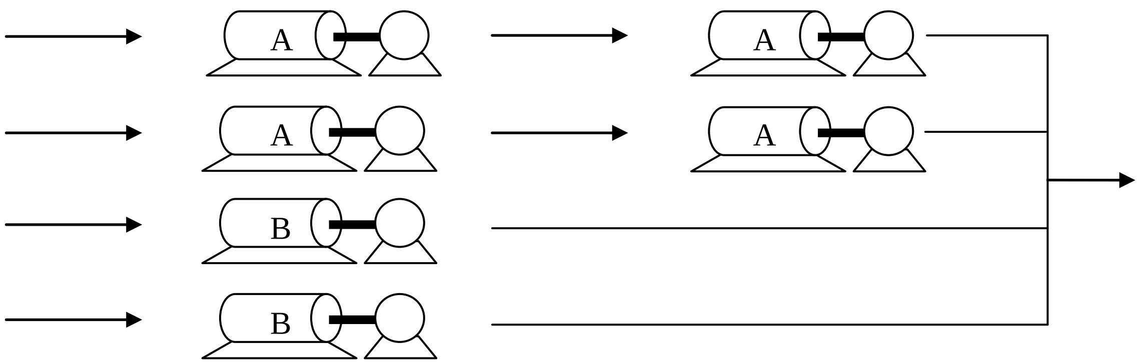
Figure 11 Mannum Pipeline profile from Crawley (1995)

5.1.4 Millbrook Subsystem

In addition to the Mannum-Adelaide pipeline, the Anstey Hill treatment zone is served by the Millbrook pipeline, which is located down-stream of the Millbrook Reservoir and aims to supplement the Mannum pipeline during peak demand.

The operation of the Millbrook pumping station is similar to that of the Mannum pumping stations with the exception of the configuration of the pumps. Shown below in Figure 12, the pumping station is four sets of pumps connected in parallel, but only two of these set have a second set connected in series. The pumps connected in series provide 1.25 GL/month of water (type 'A'), while the single pumps provide 2.5 GL/month (type 'B').

Figure 12 Millbrook Pumping Diagram



5.1.3.1 Component Reliability

The critical components of the headworks have been identified in chapter 4 and the estimates of their reliability parameters are presented in Table 16 and Table 17.

Table 16 Components in the Adelaide Mannum Pumping System

Location	Component Description	Mean Repair Time (days)	Failure Frequency (years)	Daily Availability
Mannum-Adelaide Pump Station 1	Grid Feed- Electricity			0.99965
	Pump station Transformers	7	1 in 1600	0.999988022
	Pump Station's Main Circuit Breaker	7	1 in 500	0.999961669
	Power Supply's Main Circuit Breaker	7	1 in 500	0.999961669
	Internal High Voltage Cables	7	1 in 20	0.999041732
	Switchboard	7	1 in 2000	0.999990417
	Pumps			0.998
	Supply and Intake Valves			0.998941191
Pump Station 2	Grid Feed			0.9965
	Pump Station's Transformers	7	1 in 1600	0.999988022
	Pump Station's Main Circuit Breaker	7	1 in 500	0.999961669
	Power Supply's Main Circuit Breaker	7	1 in 500	0.999961669
	Internal High Voltage Cables	7	1 in 20	0.999041732
	Switchboard	7	1 in 2000	0.999990417
	Pumps			0.998
Pump Station 3	Grid Feed- Electricity			0.99965
	Pump Station's Transformers	7	1 in 1600	0.999988022
	Pump Station's Main Circuit Breaker	7	1 in 500	0.999961669
	Power Supply's Main Circuit Breaker	7	1 in 500	0.999961669
	Internal High Voltage Cables	7	1 in 20	0.999041732
	Switchboard	7	1 in 2000	0.999990417
	Pumps			0.998

Table 17 Components in the Millbrook Pumping System

Location	Component Description	Mean Repair Time (days)	Failure Frequency (years)	Availability
Millbrook Pump Station	Grid Feed- Electricity			0.99965
	Pumping Station's Transformers	7	1 in 1600	0.999988022
	Pumping Station's Main Circuit Breaker	7	1 in 500	0.999961669
	Power supply's Main Circuit Breaker	7	1 in 500	0.999961669
	Switchboard	7	1 in 2000	0.999990417
	Pumps			0.998

Using the results obtained for the critical component's reliability, a Monte Carlo simulation was undertaken for the three pumping stations that make up the Mannum Pipeline and the single pumping station of the Millbrook supply. To produce the state transition tables shown in Table 18 and Table 21. These tables show the probability that the system will be in a particular state, which represents a discrete capacity, based on the number of pumps operating.

5.1.5 Comparison

The results of the Mannum-Adelaide Pipeline are shown in Table 18

Table 18 Pumping System Capacity and Availability

State	Percentage of output capacity	Capacity (GL/Month)	State Probability
1	100%	10.4	**0.93126**
2	86%	7.8	**5.69×10^{-2}**
3	57%	5.2	**5.00×10^{-4}**
4	29%	2.6	**$<10^{-6}$**
5	0%	0	**1.13×10^{-2}**

A frequency-duration approach and a Monte Carlo simulation were carried out on the same pipeline systems by Crawley (1995), although the approach and the system components were similar, different values were attributed to their reliability and some components were not consider by Crawley, such as drought. This produced the results presented in Table 19 and Table 20.

Table 19 Frequency-duration analysis for Mannum Pipeline from Crawley (1995)

State	Rate of Entry	Rate of Departure (Up)	Rate of Departure (Down)	State Probability
1	0.201754	0	0.006900	**0.923465**
2	0.159017	0.068966	0.001803	**6.94471×10^{-2}**
3	0.214264	0.143845	0.002095	**4.330009×10^{-5}**
4	0.000381	0.214169	0.001906	**7.65494×10^{-8}**
5	0.007047	0.142780	0	**7.04529×10^{-3}**
				1.000000

Table 20 Monte Carlo Simulation for Mannum Pipeline from Crawley (1995)

State	Cumulative State Duration (days)	Percentage Availability	Mean State Duration (Days)
1	672416	**0.9230394**	149.06141
2	49816	**0.068187**	14.18047
3	5	**0.000007**	1.66667
4	0	**0**	0
5	8337	**0.011412**	7.05927
	730574	**1.000000**	

Similar data is also presented for the Millbrook pumping station in Table 21, followed by those calculated by Crawley Table 22 and Table 23.

Table 21 Millbrook Pipeline - Monte Carlo Simulation

State	Percentage of output capacity	Capacity GL/Month	Availability
1	100%	7.5	**0.98804**
2	83%	6.25	**0.007946**
3	66.66%	5	**0.003915**
4	50%	3.75	**3.29023×10^{-5}**
5	33%	2.5	**1.64512×10^{-5}**
6	16.67%	1.25	**0**
7	0	0	**4.9353×10^{-5}**

Table 22 Frequency-duration analysis for Millbrook Pipeline system from Crawley (1995)

State	Rate of Entry	Rate of Departure (Up)	Rate of Departure (Down)	State Probability
1	0.285043	0	0.001219	**0.983557**
2	0.072540	0.071389	0.001027	**0.529301×10^{-2}**
3	0.216282	0.071341	0.000835	**0.106074×10^{-1}**
4	0.144130	0.142683	0.000642	**0.570453×10^{-4}**
5	0.071970	0.142779	00000450	**0.286378×10^{-4}**
6	0.000767	0.213976	0.000258	**0.153701×10^{-6}**
7	0.000590	0.144723	0	**0.456970×10^{-3}**
				1.000000

Table 23 Monte Carlo Simulation for Millbrook Pumping Station from Crawley (1995)

State	Cumulative State Duration (days)	Percentage Availability	Mean State Duration (Days)
1	719379	**0.984004**	823.08810
2	3902	**0.005337**	13.73944
3	7368	**0.010078**	13.54412
4	60	**0.000082**	12
5	4	**0.000005**	2
6	0	**0**	0
7	360	**0.000492**	6.79245
	731073	**1.000000**	

The Millbrook pumping station provides an over capacity to the pipeline, which connects the pumps to the water treatment plant. This limits the capacity of water provided by the pumps to 6.25 GL/month instead of 7.5 GL/month.

A comparison of the various pumping states for both the Millbrook and the Mannum pipeline shows that the values from Crawley (1995) and the values calculated here are similar.

The slight variation in the failure properties between the values found in Crawley (1995) for the frequency-duration method and the Monte Carlo method are attributable to sampling and rounding errors within the models. Since the report was written by Crawley in 1995, computational power has increased markedly and it is now possible to increase the sample size of the Monte Carlo method and reduce the rounding errors present in the report, although some will still be present.

Table 24 Comparison of Monte Carlo vs. Crawley's frequency-duration

Difference % (absolute)	State	Crawley (Mannum Monte Carlo)	Mannum (based on collected data)
Crawley (Mannum frequency-duration)	1	0.33% (0.003071)	0.83 % (7.7×10^{-3})
	2	1.81% (0.0694471)	-21.76 % (-1.24×10^{-2})
	3	83.73 % (3.6×10^{-5})	90.99 % (4.34×10^{-4})
	4	100% (-7.65×10^{-8})	97.45 % (2.92×10^{-6})
	5	-61.98% (0.00436)	37.75 % (4.27×10^{-3})

Table 25 Comparison of Crawley for the Millbrook Pipeline vs. Collected Data

Difference % (absolute)	State	Crawley (Millbrook Monte Carlo)	Millbrook (based on collected data)
Crawley (Millbrook frequency-duration)	1	0.0454% (4.47×10^{-3})	-0.45% (-4.43×10^{-2})
	2	0.83% (4.40×10^{-5})	-49.84% (-2.26×10^{-2})
	3	-4.99% (-5.29×10^{-4})	62.53% (6.63×10^{-2})
	4	43.74% (2.49×10^{-5})	45.66% (2.60×10^{-5})
	5	-82.54% (-2.36×10^{-5})	82.54% (-2.36×10^{-5})
	6	-100% (-1.54×10^{-7})	100% (1.54×10^{-7})
	7	7.67 % (3.50×10^{-4})	85.11% (3.80×10^{-3})

The data is most relevant around the first pumping state, where all the components are working, as this represent the largest proportion of the total reliability. When compared, these values differ by 0.83% (0.0031) and 0.43% (7.7×10^{-3}) from the frequency-duration approach preformed by Crawley. Clearly, there is very little difference between the values, and the probability of the system not providing water to Adelaide is very low.

The major source of the variation between Crawley's results and the values in this report is due to the additional parameters for drought which is applied to the river intake, and variations in the reliability of the components selected.

The above analysis stops at the pumping stations and does not cover the connection from the treatment plant to the point of use. This was due to Crawley's research focusing on the bulk transfer of water. To determine the reliability of the supply connection at the premises, the connections between the pumping stations and premises need to be considered.

Of the components noted in chapter 4, only the pipeline line itself and the final storage tank would have an effect on the reliability of the system. The final storage forms part of the distribution and treatment system, which is connected after the pipelines join. The availability of components in Anstey Hill distribution and treatment appear in
Table 26.

Table 26 Anstey Hill Treatment Components

Location	Component	Mean Time to fail (days)	Frequency (1/years)	Availability
Post pipelines	Distribution			0.999954
	Anstey Hill Tank	3	50	0.999840
	Pipes			0.999986

The two pumping system operate in parallel (shown in Figure 8), up to the storage tank, and then the water is filtered through a single water treatment plant. With the addition of these components, plus the reticulations system, the reliability at the boundary of a property in Adelaide covered by the Anstey Hill water treatment plant is shown in Table 27.

Table 27 Anstey Hill Availability

	Availability of Water Supply
Anstey Hill	99.86% (0.9986)

5.2 Hope Valley

The Hope Valley reservoir is primarily used as a service reservoir from which water is distributed to the CBD and surround areas of Adelaide from the two larger reservoirs located on the Torrens subsystem. This distribution is conducted off-stream; water is piped into the Hope Valley reservoir from the other dams, with water from the local catchment prevented from entering the reservoir.

These off-stream connections to the Millbrook and Kangaroo Creek Reservoirs and the connection from the Mannum pipeline, negate the effect that any single reservoir failure would have on the system. The Hope Valley reservoir is therefore treated in a similar method to the Little Para reservoir; as a single reservoir.

Table 28 Hope Valley

	Availability of Water Supply
Hope Valley	99.25% (0.9925)

5.2.1 Summary of the Northern Adelaide Metropolitan Supply

Based on the data obtained for the metropolitan water supply of Adelaide, there are three main supply cases: a multi reservoir system with a redundant reservoir supply, a pumping station system that draws water from both a reservoir and a river supply and a single reservoir system. The availability of water, the percentage of time that a property will have water, when provided with only a single town's main connection is approximately:

Table 29 Summary of Results

Location	Availability
Barossa/Little Para	99.99%
Anstey Hill	99.86%
Hope Valley	99.25%

It is not surprising the Barossa/Little Para region has the lowest disruption to the supply. With a supply fed by gravity from two separate water sources, and treated by two separate treatment plant minimising the chance that any one failure can disrupt the system.

The reliance of Anstey Hill on pumping stations increases the probability of disruptions to the water supply. This is despite the large number of redundant pumps and an alternative supply. The increase is largely due to the vulnerability of pipeline connections between the pumping stations, and the use of only a single treatment plant to filter the water.

With only a single supply connected to the town mains, it is not surprising that the Hope Valley area has more disruptions than the other regions. The lack of any redundancies between the source and treatment exposes the Hope Valley region to prolonged outages, due to supply or storage failure.

The intermediate results for the Millbrook pumping station, noted in section 5.1.5, are similar. However, Crawley in his report did not consider failure of reservoirs and only looked at the continuity of the water supply from these reservoirs.

Similarly, the values obtained for the reliability Barossa/Little Para system are less than those used by Feeney and Thomas for the water supply reliability. Again, this difference is largely due to the inclusion of reservoir failures with this report.

5.3 Selection of Town's Main Scenarios for Further Analysis

Most cities and towns throughout the world have water supply infrastructure consisting of the same basic components that are constructed into a complex network consisting of: water source, pumping stations, pipeline, reservoir, filtration, and storage. This section of the report aims to construct a generic model of these systems, which represents the water supply infrastructure present in most New Zealand cities.

An examination of the water supply infrastructure throughout Australia and New Zealand leads to the formulation of four basic scenarios, which are common to the water supply systems. These scenarios represent an idealised model making it applicable as a generic solution. The four scenarios are:

- Single reservoir (Figure 13)
- Dual reservoirs, (Figure 14)
- Pipeline pumping water from wells or rivers, (Figure 15)
- Pipeline pumping water from wells or rivers in parallel with an elevated dam system (Figure 16)

The entire city's water supply network tends to be much more complex than this. However, the supply to an area of the city is limited to the region covered by a water treatment plant.

The inclusion of these single pumping case scenarios was to provide the worst real world supply condition. The real world scenarios favour the town's main supplied from multiple-reservoir systems.

For the pumping system, an output of 50 % of greater is considered a working state. This selection is based on the review of average daily usage vs. the potential peak throughput of pumping systems.

5.4 Sensitivity of the components

The components values and assumptions within the model are subject to change and error. The potential impact these errors and assumptions have on the model are investigated through a sensitivity analysis.

The sensitivity analysis within this report is conducted by varying a single major component away from the values found within the research and observing the change this has on the system reliability for each of components within the four town water scenarios.

The rounding error present within the Monte-Carlo model leads to a larger range of variation being used than would be typical. The larger range allows for the trends within the model to be observed, whilst minimising the variation from the rounding error.

5.4.1 Single Reservoir Scenario

Figure 13 shows the schematic of the single pumped system. From the schematic it can be seen that four main variables will effect the reliability of a system constructed of a single reservoir:

- The supply reliability (reliability of there being water within the reservoir excluding structural failures),
- Reservoir failure or a structural failure of the reservoir,
- Pipe damage between the reservoir and the treatment plants,
- Treatment plant and the distribution or reticulation reliability.

The typical value for all five of these components resides around an availability of 0.9999, with the exception of the pipe failures, which varies with the length of pipe.

The graph below shows the sensitivity of the system to individual variation of these five components

Figure 13 Schematic of Single Reservoir

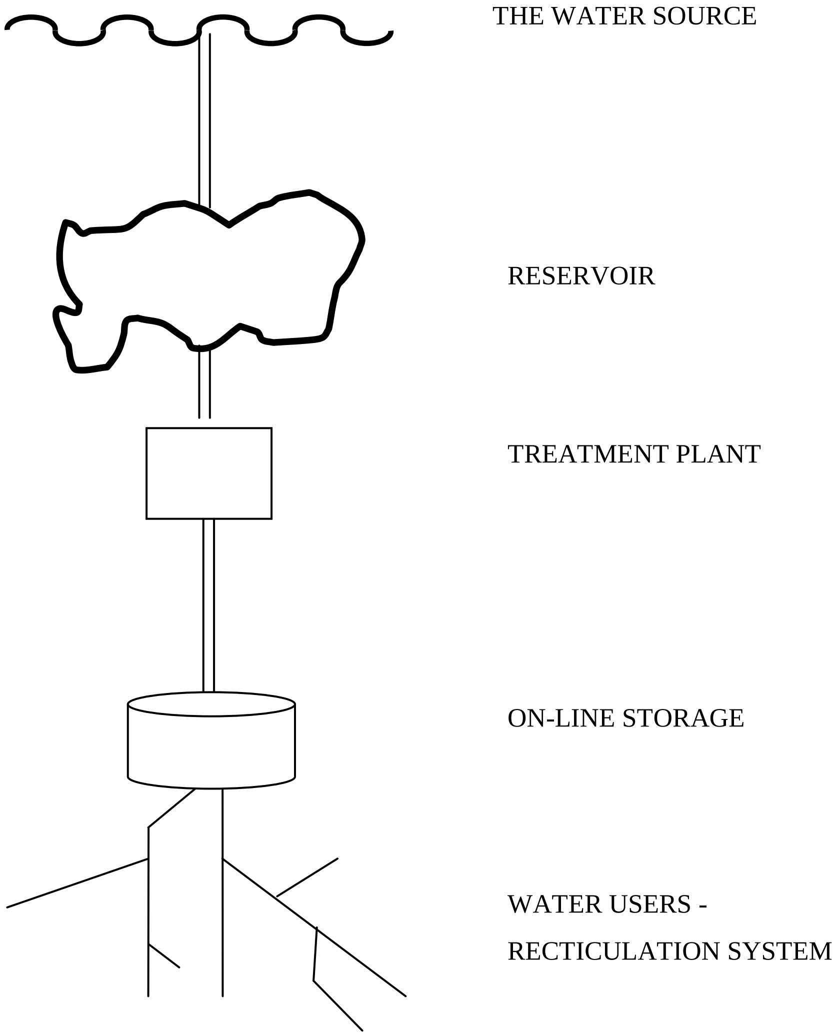
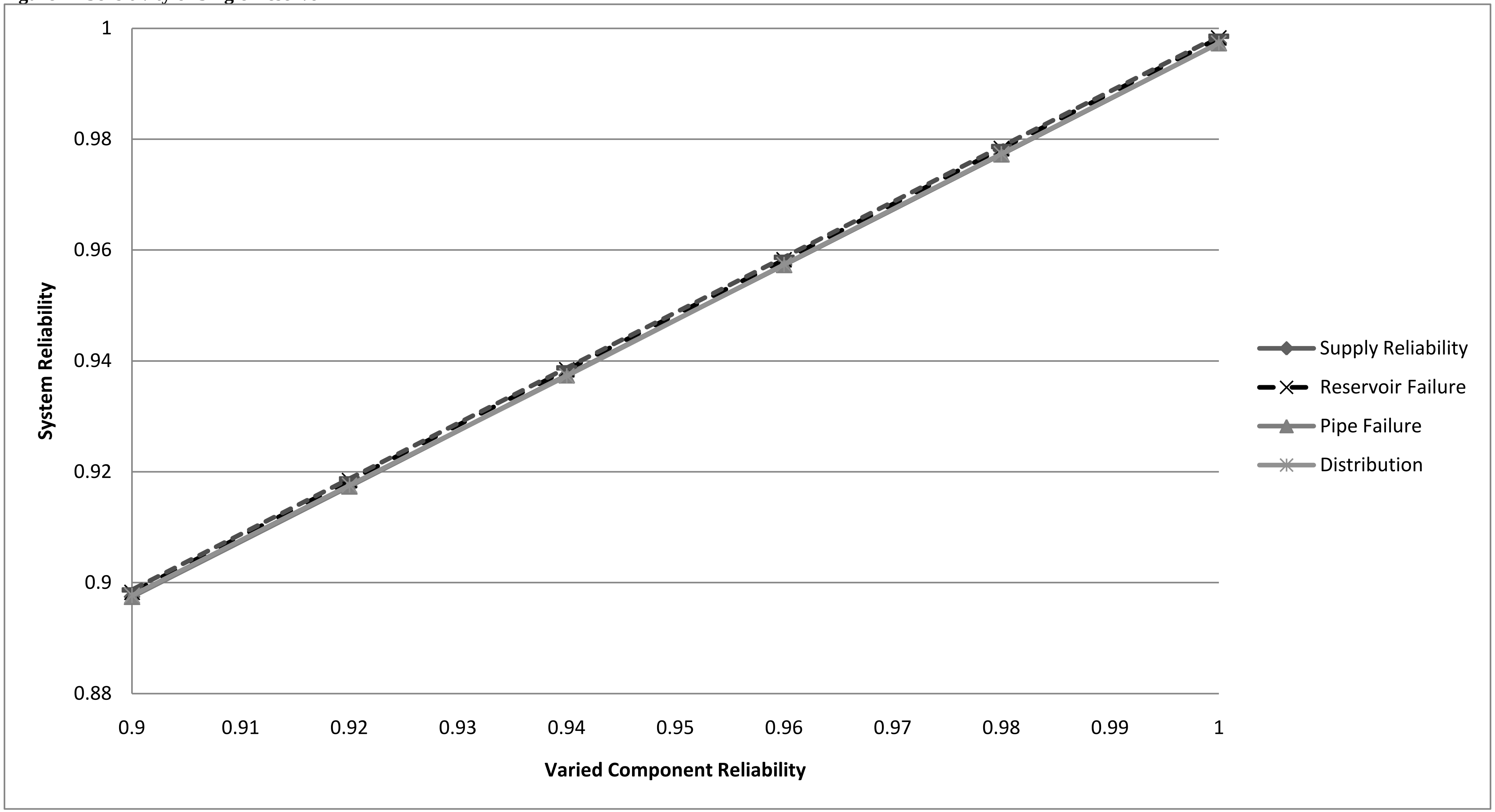


Figure 14 Sensitivity of Single Reservoir



The following observations can be made about the single reservoirs system components in the sensitivity analysis:

- It is the simplest scenario
- Connected in series, variations in the reliability of the components produces a linear relationship for all the components.
- All of the components have an equal significance in the single reservoir scenario.

5.4.2 Dual Reservoir Scenario

The dual reservoir system, shown is Figure 15, has the same main components as the single reservoir system. However, for the dual reservoir scenario all the components, with the exception of the reticulation system, have a redundant component connected in parallel. As noted in the single reservoir section the base values for these components from the research is approximately 0.9999. The exception is the pipe which is varied based on length, but for typically lengths of pipe this offers an additional order of magnitude of reliability (0.99999).

When compared to the single reservoir graph the dual reservoir exhibit a greater level of reliability. This is expected, as the major components of the system have greater redundancy provided by the second reservoir.

From the graph in Figure 16, it can be seen that a number of the lines will overlap when plotted. This is due to the similarity of their base values and method of connection. The exception to this is the reticulation which is connected to the merged system and exhibits a linear variation in the sensitivity plot.

Therefore, the significance of the reticulation system outweighs that of any other component.

Figure 15 Schematic of Dual Reservoirs

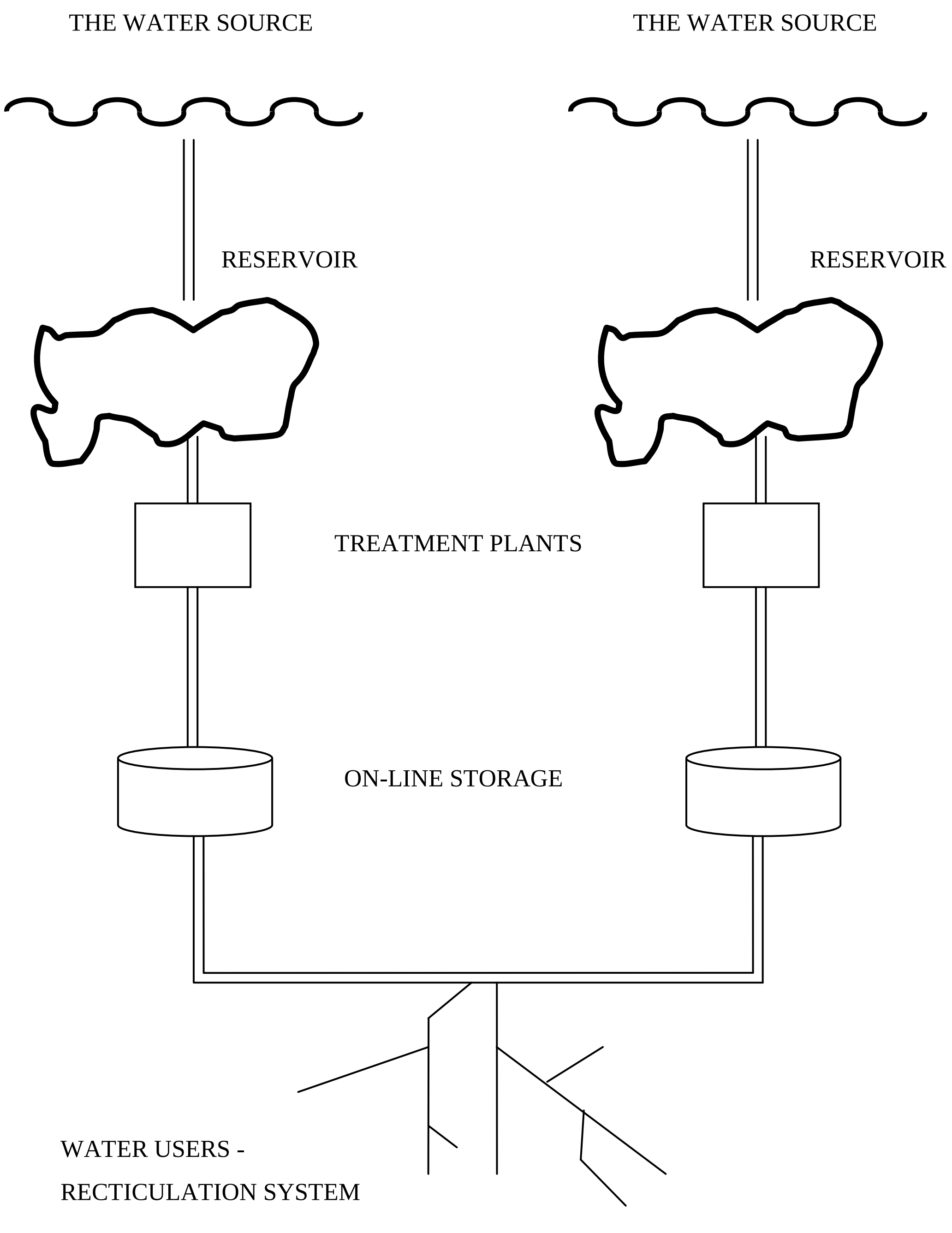
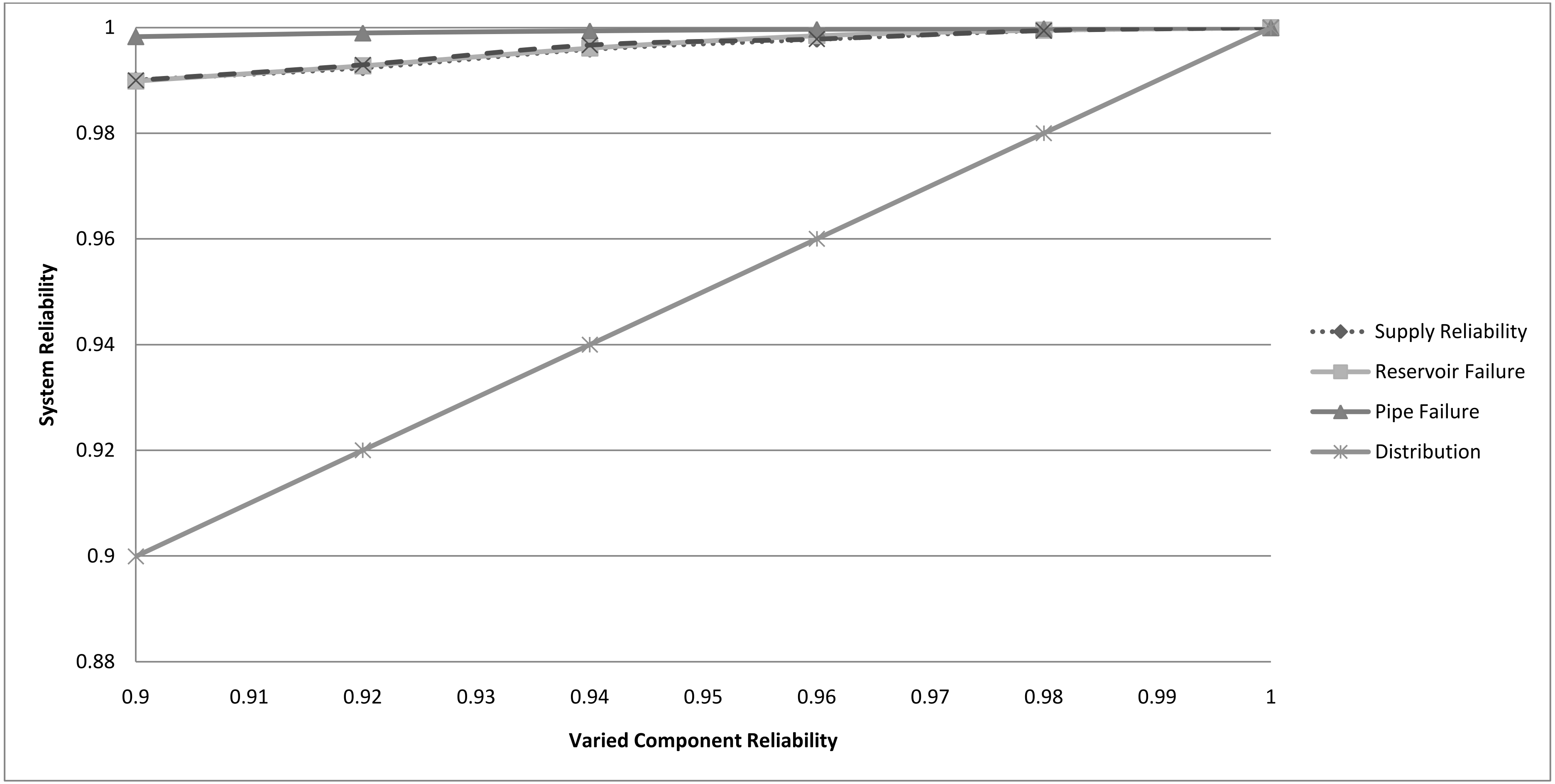


Figure 16 Sensitivity of Dual Reservoir



5.4.3 Single pumped pipeline

The single pumped pipeline, shown in Figure 16, is a more complex system than that of the reservoir supplied water. This complexity is due to attempts to limit the effect that the pumps have on the system reliability by providing over capacity and redundant connections at the pumping stations.

Typically, water is pumped up and out of the source to another pumping station, which pumps the water to tanks which then gravity feeds the reticulation system. This results in six main components being connected to a single pumped pipeline:

- Supply reliability (reliability of there being water within the source),
- Pump reliability,
- Electrical components associated with the pumps,
- Pipe failures (between pumps, treatment plant, source, and tank),
- Water tank
- Distribution or reticulation reliability.

From Figure 18 it can be seen that variations in the pumps and motors reliability has the least variation on the system reliability. This is due to the redundancy of multiple pumps acting in parallel at each of the pumping stations. The rapid drop-off in the system reliability is due to variations in pumps reliability and is attributed to the pumps being connected in series for the water extraction and horizontal pumping. This combines with the parallel connections and 50% over capacity at each of the pumping station to give the line the curved shape.

The pipe network causes a rapid drop-off in reliability. This is due to a single failure in the pipe causing a complete failure of the system and the multiple occurrences of single pipes between pumping stations, tanks, source, and reticulation.

Water treatment, tank, distribution (reticulation) all remain linear as they are connected in series past the network of pumps and electrical components.

The electrical components consist of a number of elements connected in series. These components were grouped together to examine their effect, as a whole, on the system. If they were assessed individually, there would a similar linear relationship as the pipe failures. However, as a whole they cause the system's reliability to decrease in a rapid fashion due to multiple series connections, and have the most effect on the system reliability when varied.

Figure 17 Schematic of Single Pumped Pipeline

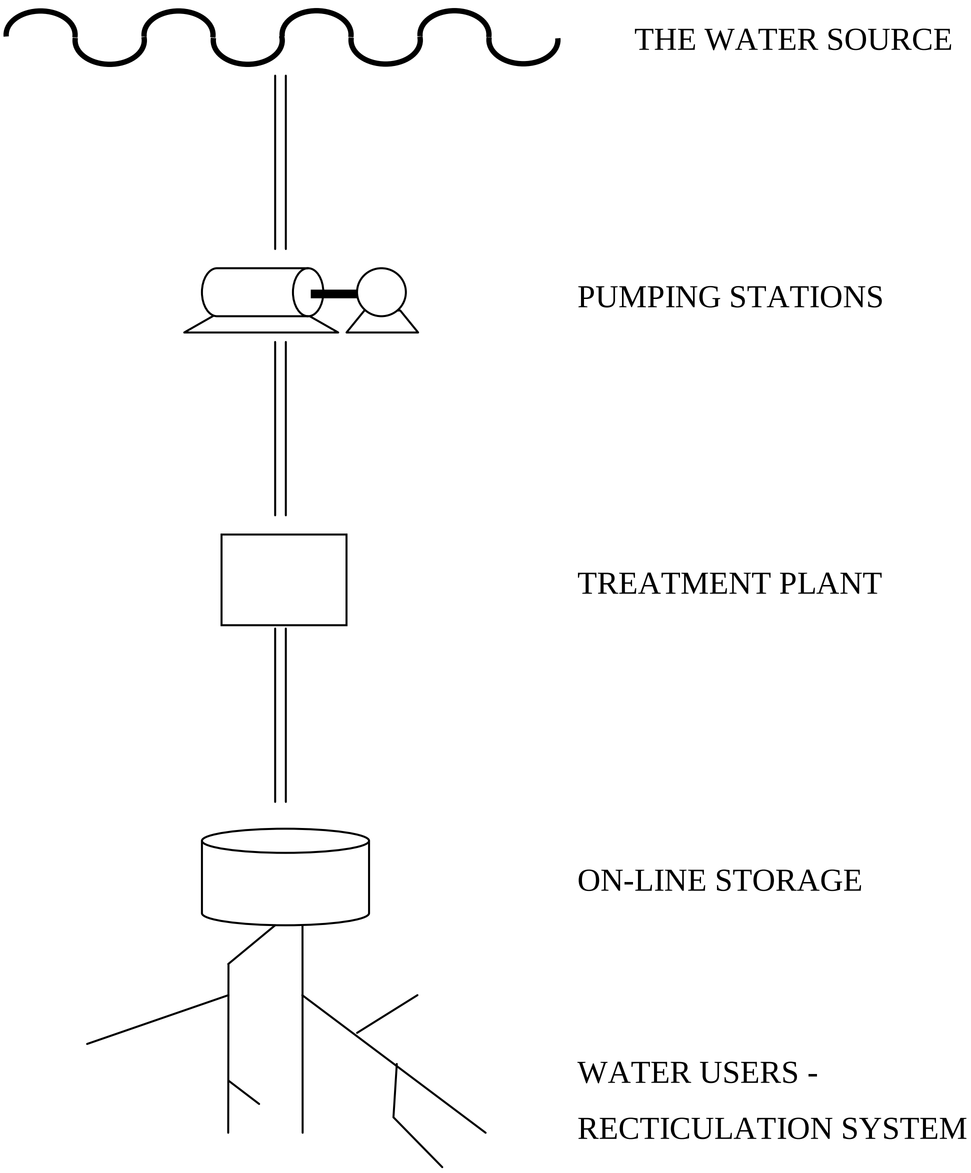
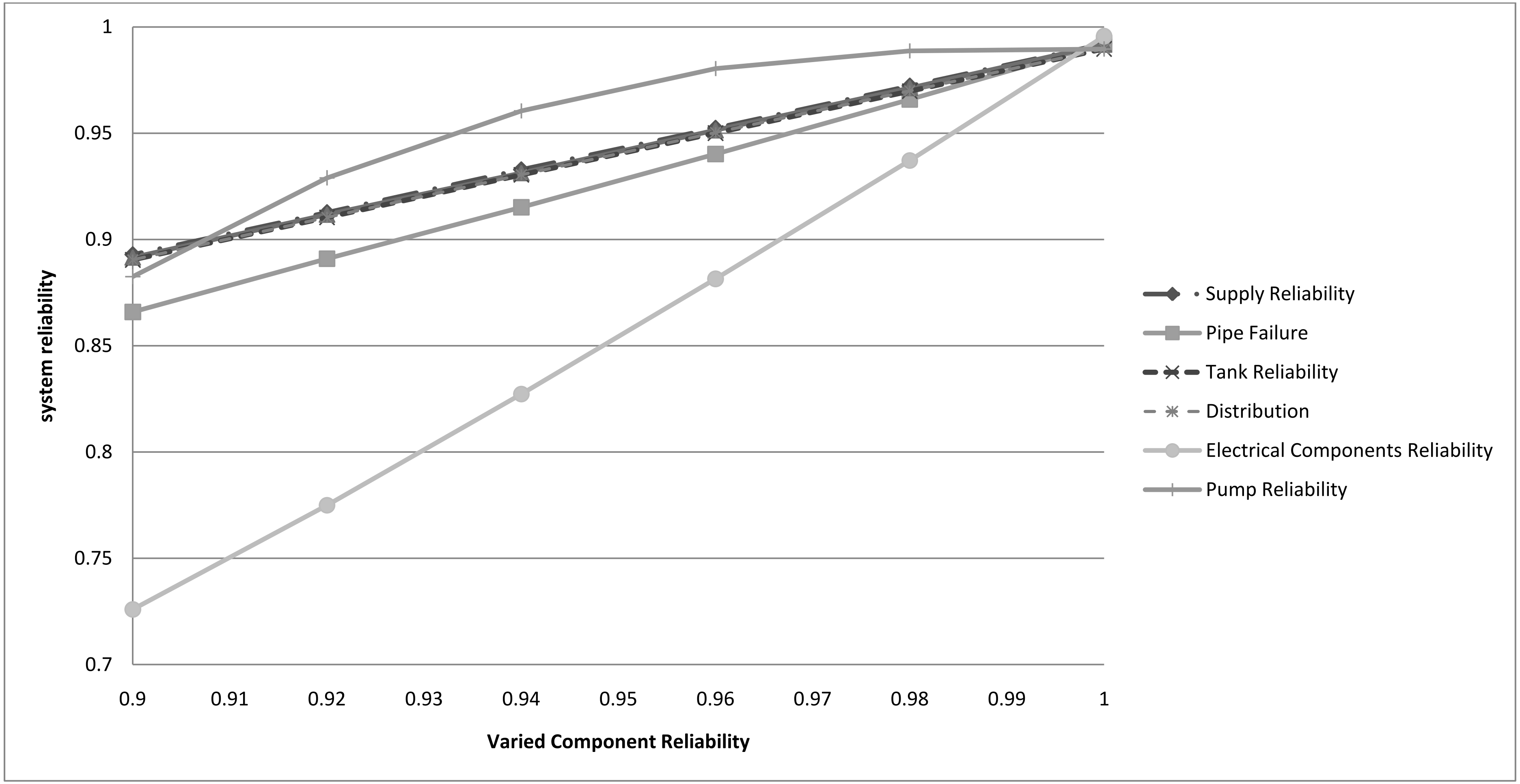


Figure 18 Sensitivity of Single Pumped Pipeline



5.4.4 Combined Pumping Station and Reservoir Scenario

As this scenario is a combination of a single pumping system and a single reservoir system, combined at a point prior to the water treatment plant, the components of the systems are a mix of the two scenarios.

- Supply reliability (reliability of there being water within the source),
- Pump reliability,
- Electrical components associated with the pumps,
- Pipe failures (between pumps, treatment plant, source, and tank),
- Water tank
- Distribution or reticulation reliability.
- Reservoir failure or a structural failure of the reservoir

The interaction between the components of the pipeline and reservoir system is much more complex than any of the other scenarios.

Figure 19 shows the schematic of the combined pumping and reservoir system. It can be seen that all the components connected before the system merges into the water treatment plant are connected in parallel. As expected they have a greater reliability than the single reservoir or pipeline scenarios.

Those connected after the merger; i.e. water treatment plant, tanks, and reticulation, exhibit a linear variation. This linear variation is a result of their serial connections. This means that variations in the components reliability will have an equal effect of the system reliability.

Electrical components have a sharp curve as noted above. There are a large number of serial connections between the electrical components and in this case, they are connected in parallel to the alternative supply. This parallel connection produces the curve in the line not exhibited in the single pipeline scenario.

Due to the redundancy provided by the pumping system, an increase in the probability of failure of the reservoirs, pumps, and supply (drought) have little effect on the system.

Figure 19 Schematic of Combined Pumping Station and Reservoir.

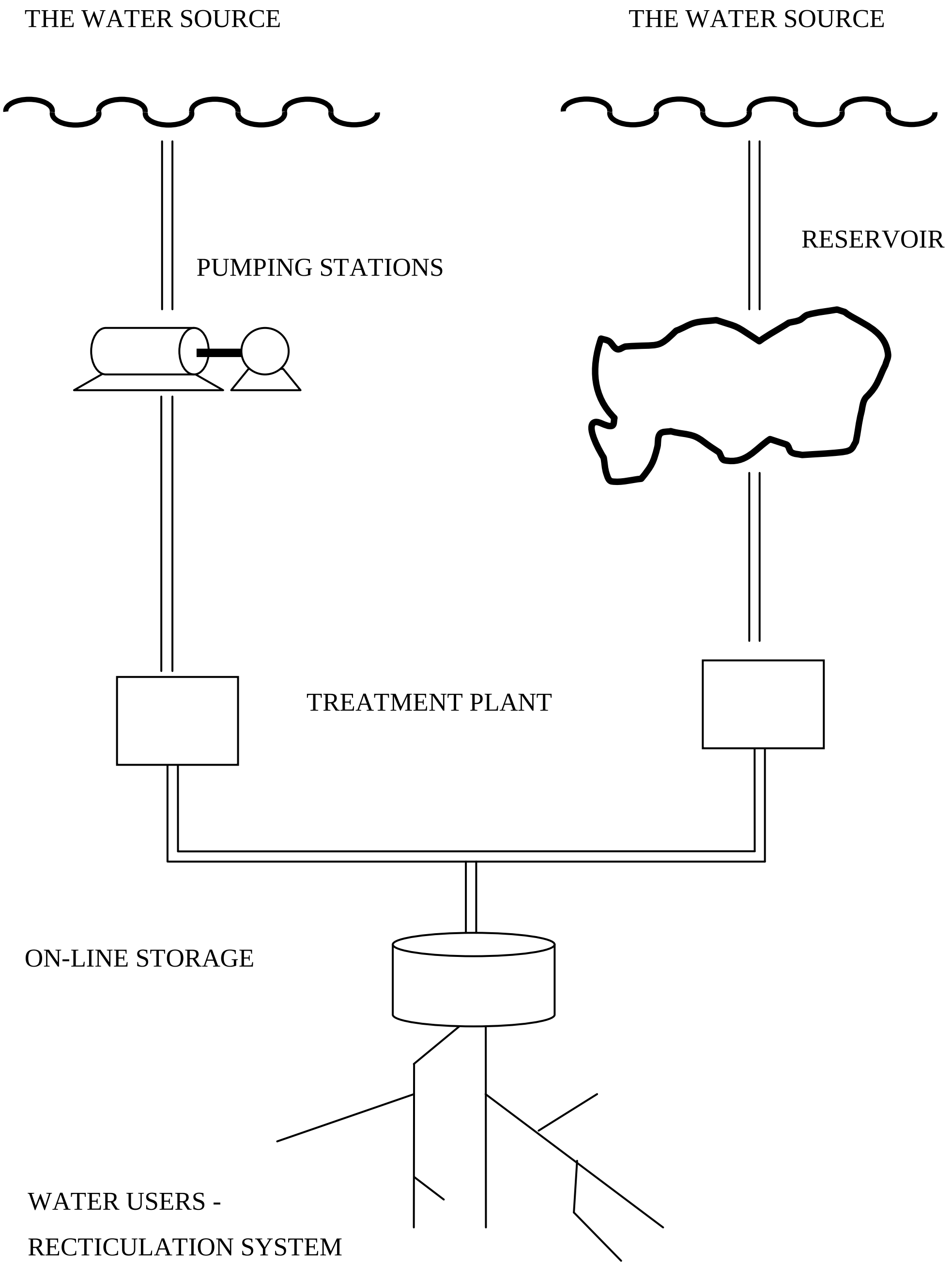
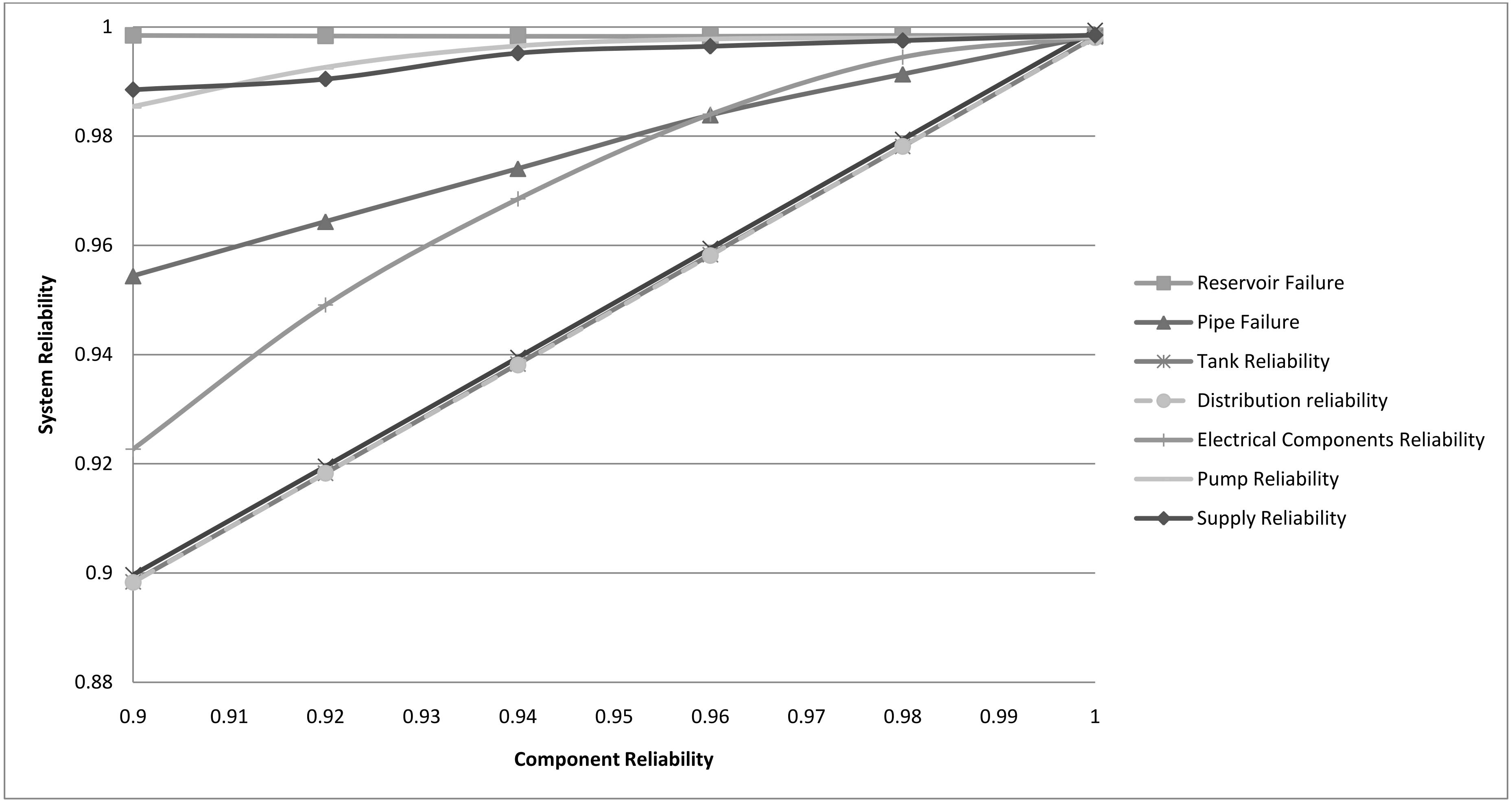


Figure 20 Sensitivity of Combined Reservoir and Pumped Pipeline



5.5 Summary of the reliability of the town's main.

A summary of the availability of the four scenarios considered is provided in Table 30 Summary of scenarios.

Table 30 Summary of scenarios

Scenario	Availability
Single Reservoir	0.99862
Dual Reservoir	0.99996
Single Pipeline	0.99092
Combined Pipeline and Reservoir	0.99468

The reliability of water supplied by the dual reservoir is superior to the other supplies by significant margin. While, the single pipeline is the least reliable of the supplies considered.

Chapter 6. Analysis of Secondary Water Supplies

This chapter takes the values collected in Chapter 5 for the reliability of town's main and the values collected in Chapter 4 for the reliability of the individual components and combined them based on the four selected scenarios to produce results for the three classes of water supply shown in Figure 21 and Figure 22. The objective is to differentiate between the reliability of the different types of connections allowed under the New Zealand sprinkler standard.

There have been some assumptions made about the nature a main's water supply system; these were provided to give an estimate of the range of supply available within a generic town. The occurrence of these supply favours the multi-dam and pipeline-dam systems over those of the single connections. These generic scenarios have also been made to avoid the need to specifically assess all the interconnections between wells, dams, and rivers – a simplification necessary to be able to derive general conclusions about the system reliabilities.

The discussion so far has focused on the reliability of the water supply within the town's supply and the components that make up the secondary supplies. These can now be combined to determine the reliability of the different connection types discussed in Chapter 2.

Based on the result of these combinations, the probability of a water system designed to use a Class A or Class B2 supply from the New Zealand sprinkler standard has an availability within the range shown in Figure 21. This high range of availability is due to the high level of reliability in each of the components in the water supply system and the redundancy of the secondary water supplies.

To illustrate just how reliably these supplies are: the annual probability of failure of a town's main supplied by dual reservoirs is in the order of 5 ¼ minutes per year (i.e. $\sim 1\text{x}10^{-5}$). This probability can be approximated, as being between struck by lightning in one's lifetime is 1.6×10^{-4} (National Weather Service) and winning the lottery with single entry ($\sim 1 \times 10^{-7}$) lotto.

Figure 21 The System Reliability for Dual Water Supplies

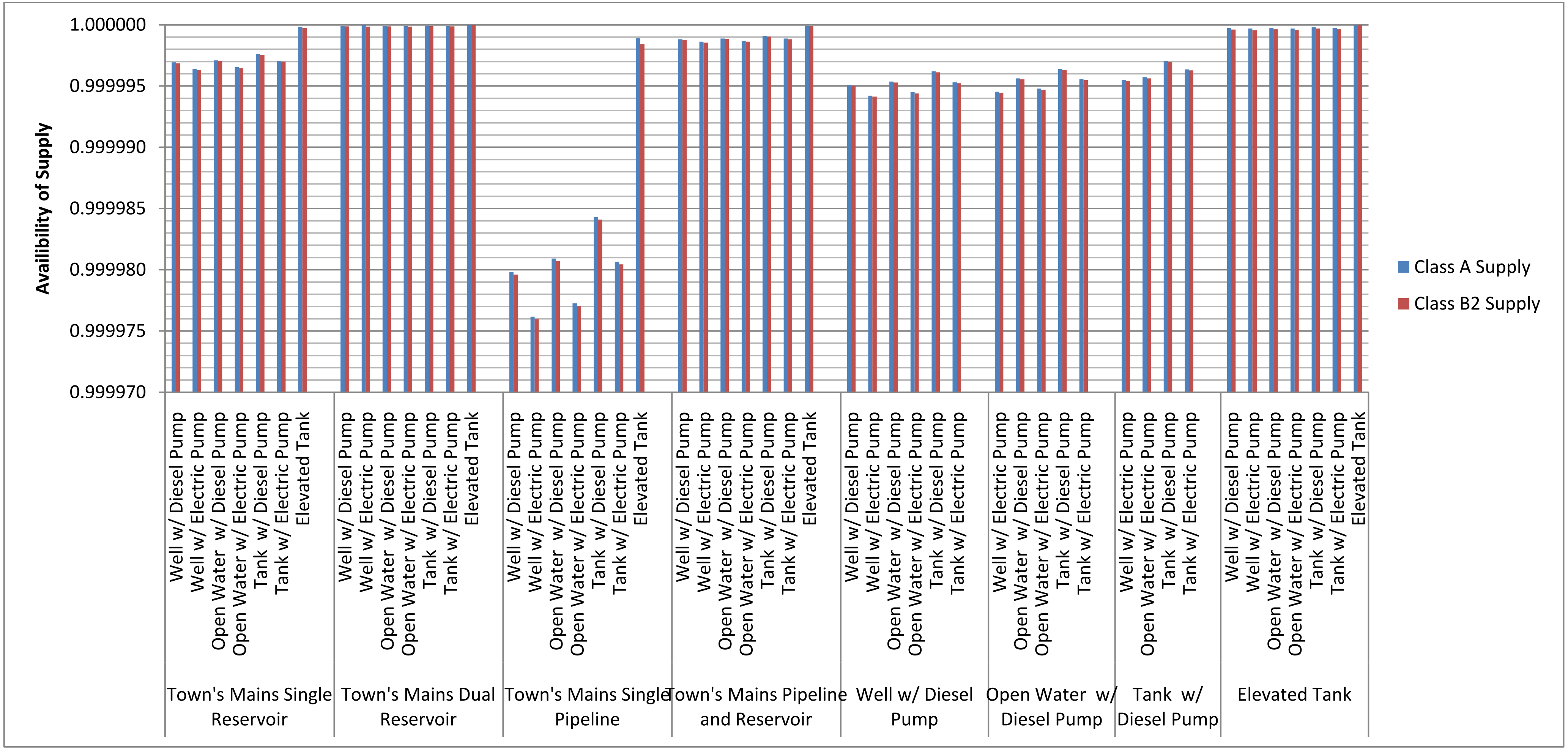


Table 31 Reliability of Water Supplies

Primary	Secondary	Class A	Class B2
Town's Mains Single Reservoir	Well w/ Diesel Pump	0.999996926	0.999996855
	Well w/ Electric Pump	0.999996370	0.999996291
	Open Water w/ Diesel Pump	0.999997092	0.999997023
	Open Water w/ Electric Pump	0.999996536	0.999996460
	Tank w/ Diesel Pump	0.999997610	0.999997549
	Tank w/ Electric Pump	0.999997054	0.999996985
	Elevated Tank	0.999999833	0.999999757
Town's Mains Dual Reservoir	Well w/ Diesel Pump	0.999999912	0.999999867
	Well w/ Electric Pump	0.999999995	0.999999843
	Open Water w/ Diesel Pump	0.999999917	0.999999874
	Open Water w/ Electric Pump	0.999999901	0.999999850
	Tank w/ Diesel Pump	0.999999932	0.999999896
	Tank w/ Electric Pump	0.999999916	0.999999872
	Elevated Tank	0.999999995	0.999999990
Town's Mains Single Pipeline	Well w/ Diesel Pump	0.999979818	0.999979600
	Well w/ Electric Pump	0.999976171	0.999975945
	Open Water w/ Diesel Pump	0.999980908	0.999980692
	Open Water w/ Electric Pump	0.999977261	0.999977037
	Tank w/ Diesel Pump	0.999984310	0.999984101
	Tank w/ Electric Pump	0.999980661	0.999980445
	Elevated Tank	0.999998906	0.999998423
Town's Mains Pipeline and Reservoir	Well w/ Diesel Pump	0.999998824	0.999998769
	Well w/ Electric Pump	0.999998611	0.999998549
	Open Water w/ Diesel Pump	0.999998887	0.999998835
	Open Water w/ Electric Pump	0.999998675	0.999998615
	Tank w/ Diesel Pump	0.999999085	0.999999041
	Tank w/ Electric Pump	0.999998873	0.999998820
	Elevated Tank	0.999999936	0.999999905

Table 31 Reliability of Water Supplies Continued.

Primary	Secondary	Class A	Class B2
Well w/ Diesel Pump	Well w/ Diesel Pump	0.999995107	0.999995022
	Well w/ Electric Pump	0.999994223	0.999994130
	Open Water w/ Diesel Pump	0.999995371	0.999995289
	Open Water w/ Electric Pump	0.999994487	0.999994397
	Tank w/ Diesel Pump	0.999996196	0.999996120
	Tank w/ Electric Pump	0.999995311	0.999995228
Open Water w/ Diesel Pump	Well w/ Electric Pump	0.999994535	0.999994444
	Open Water w/ Diesel Pump	0.999995621	0.999995541
	Open Water w/ Electric Pump	0.999994785	0.999994697
	Tank w/ Diesel Pump	0.999996402	0.999996328
	Tank w/ Electric Pump	0.999995565	0.999995484
Tank w/ Diesel Pump	Well w/ Electric Pump	0.999995509	0.999995425
	Open Water w/ Electric Pump	0.999995714	0.999995633
	Tank w/ Diesel Pump	0.999997043	0.999996977
	Tank w/ Electric Pump	0.999996355	0.999996281
Elevated Tank	Well w/ Diesel Pump	0.999999735	0.999999615
	Well w/ Electric Pump	0.999999687	0.999999546
	Open Water w/ Diesel Pump	0.999999749	0.999999636
	Open Water w/ Electric Pump	0.999999701	0.999999567
	Tank w/ Diesel Pump	0.999999794	0.999999700
	Tank w/ Electric Pump	0.999999746	0.999999631
	Elevated Tank	0.999999986	0.999999970

It is clear that there is a difference between the reliabilities of Class A and B2 supplies. This results from the exposure of the system to earthquakes. Although the difference appears to be small, if they were examined after an earthquake the variation would be much larger. The values shown here only reflect the relatively rare occurrence of a major earthquake. The susceptibility of the pipe network during these earthquakes reduces the reliability of the Class B2 site main. If the earthquakes are ignored as a factor in the reliability, the availability of water supplied by a Class B2 supply would be marginally more reliably than that of the Class A supply due to the redundant pipe lines.

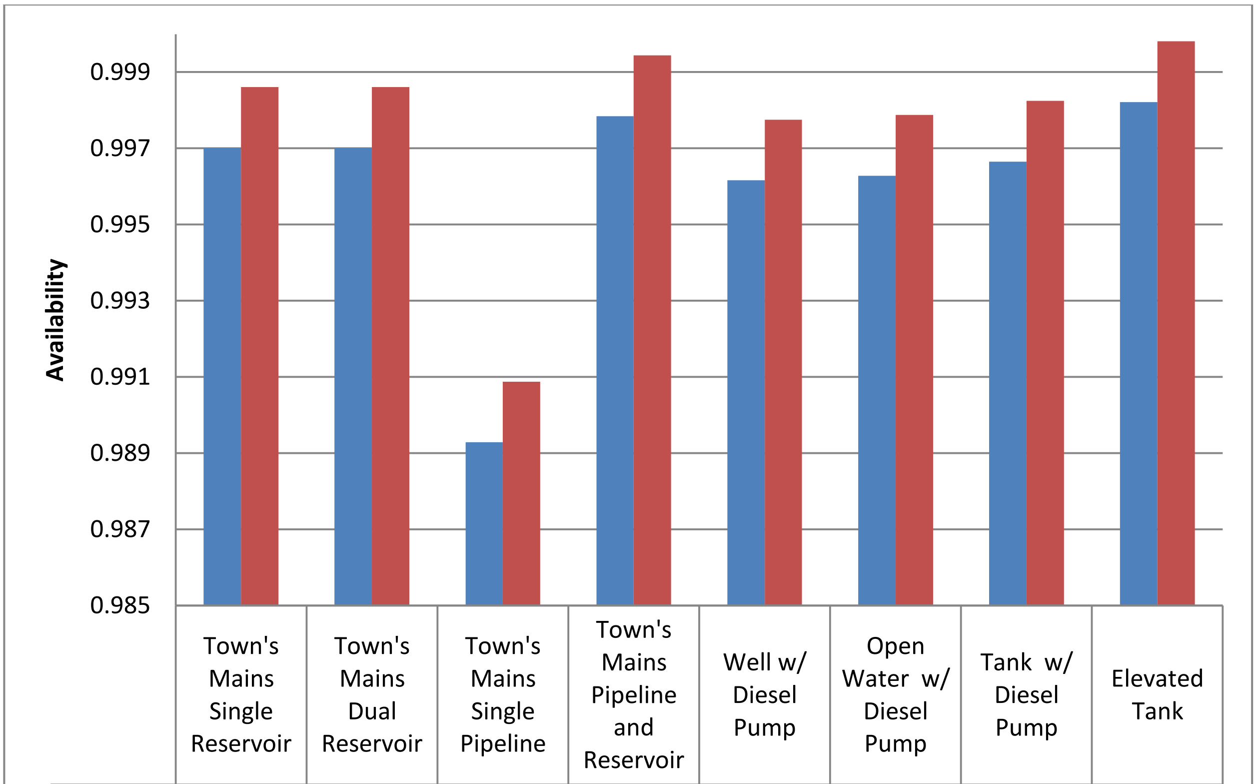
The influence of earthquakes is also shown in the difference between the reliability of open water and well water. With the slight variation in reliability seen in all Classes, A, B2 (figure 17) and C (figure 18), the result of the earthquakes having a greater effect on the well sourced water. More discussion on the influence of earthquakes follows in section 6.1.

From Figure 21, it is evident that elevated tanks offer a superior level of reliability when compared with those that require a pump to feed the water. Even when attached to the comparatively unreliable pipeline supplies. The effectiveness of gravity feeding the water is the main reason for this low probability of failure.

When the pumps are compared, both electrical and diesel pumps offer very similar levels of reliability (availability of 0.998 for diesel vs. 0.9984 for electric). Variations in the maintenance, manufacture and design of the pumps could have a greater impact on its reliability, under normal conditions, than the type of motor driving the pump. However, during a fire the isolation from outside power sources, required for electric pumps, would mean that diesel pumps would offer a more predictable failure rate. This is illustrated by the requirement of the New Zealand Standard and many others for the use of diesel pumps as the primary source of pumped water.

The reliability of these pumps is the dominate factor in the reliability of the Class C connections. This results in Class C connections having a much low reliability in general than those of either Class A or B. Class C2 supplies connected to a dual reservoir supply may result in a reliability that is close to those of a Class A or B supplies when connected to a primary supply provided by a single pumped pipeline. The probability of failure of an open water supply boosted with diesel pump and connected primary supply provided by single pumped pipeline (Class B2 supply) is 2.41×10^{-5} compared to 5.57×10^{-4} for Class C2 supply. This is the result of the single pumped pipeline supply have a much lower reliability than the dual reservoir supply. However, connections to a single pipeline supply are typical limited to those connected at the head of the water supply.

Figure 22 Class C1 & C2 Supplies



	Town's Mains Single Reservoir	Town's Mains Dual Reservoir	Town's Mains Single Pipeline	Town's Mains Pipeline and Reservoir	Well w/ Diesel Pump	Open Water w/ Diesel Pump	Tank w/ Diesel Pump	Elevated Tank
Class C1	0.997011	0.997011	0.989289	0.997846	0.996161	0.996280	0.996653	0.998216
Class C2	0.998606	0.998606	0.990872	0.999443	0.997755	0.997874	0.998247	0.999813

Even with the redundancy of the secondary supply removed, the supply of water from all the sources except the pumped pipeline still provides a very high level of availability. The probability range of the Class C1, a boosted single supply, is shown in Figure 22.

As noted, the occurrence of pumped town's main in isolation is rare, however, it can be concluded that the reliability of these pumped sources would greatly benefit from the addition of gravity feed online storage, whether it is provided on-site or within the town's supply.

The values calculated by Feeney (2001) for the reliability of sprinkler water supplies are shown in table 31.These figures were based on the previous New Zealand standard and do not include town's mains infrastructure outside the reticulation system. From the table, it is clear the reliability of the pumps dominate the single supply connections when a pump is connected. Overall, the values from Feeney and the values for the dual reservoir scenario are closely related.

Table 32 Comparison with Feeney results

Supply		Probability of no water (Feeney)	Estimate from this report for similar supplies (based on dual reservoirs)
Single Town's Main	No pump	8 $x10^{-5}$	3.20 x 10^{-5}
	Diesel pump	1.58 x 10^{-3}	1.64 x 10^{-3}
	Electric pump	2.08 x 10^{-3}	-----
Dual Town's Main	No Pumps	6.49 $x10^{-9}$	1.75 $x10^{-9}$
	1 Diesel and 1 Electric Pump	4.33 x 10^{-6}	3.35 x 10^{-6}
	1 Diesel Pump	1.50 x 10^{-3}	1.60 $x10^{-3}$
	1 Electric Pump	2.00 x 10^{-3}	2.00 $x10^{-3}$
Town Main & Tank	Town's main and diesel pumped tank	1.65 $x10^{-7}$	$8.85x10^{-8}$
	Town main and tank with electric pump	$1.25x10^{-7}$	7.18 x 10^{-8}
	Town's main with electric pump and tank with diesel pump	3.25 x 10^{-6}	2.26 x 10-5
	Town's main with diesel and tank with electric pump	3.11 x 10^{-6}	3.56 x 10^{-6}
	Town's main with diesel and tank with diesel pump	4.11 x 10^{-6}	4.32 x 10-6
	Town's main with electric and tank with electric pump	$2.46x10^{-6}$	1.83 $x10^{-5}$

6.1 *Discussion of Earthquakes*

The variation between Classes A and B2, supplied by the same source type, is limited to the effect that the earthquake has on the additional pipe work installed in the loop main of a Class B2 supply. This additional length of pipe leads to a relative small difference between the classes shown in figure 17. To further examine the effects that an earthquake has on the various classes and types of supply it is assumed that an earthquake will occur. This is done by setting the probability of an earthquake to 100% (P (earthquake) = 1) rather than using a return period (P(earthquake) =0.0021).

In Table 33 it can be seen that there is a significant change in the probability of failure of the water source when compared to those discussed in Chapter 4. This of course is to be expected as discussion in Chapter 4 showed that the water supplies are vulnerable to damage from liquefaction, slumping, and permanent transformation of the ground. These factors are largely depended on the type of soil at site of the earthquake and the response that these supplies have to ground movement.

Table 33 Probability of failure after an earthquake

Source	Probability of Failure
Tanks	0.131344
Pumping Station Failure	0.909275
Well/Bore	0.761302
Surface Water	0.453025

The integrity of water tanks is likely to be high. The likely construction methods for water tanks, reinforced concrete and steel, have been shown to survive earthquake damage (Knoy 1996) when design for seismic forces. The analysis leads to the lowest probability of failure (approximately 13% during earthquakes), this in part due to the design but is also the result of the amount of water that is stored. In the period following the earthquake sufficient water may remain in the tank to provide a reliable source of water. This is due to leaks slowly draining the tank.

This is contrast by the rapid and high probability of failure of the pumping stations. This is attributable to the high frequency of failure during earthquakes of mains power supplies resulting in failure of the electric pumps.

Unlike the pumping stations, underground water supplies need not be dependent on the electricity to ensure a continued supply. However, underground water sources are susceptible to failure from ground movements causing sanding or structural failure of the well/bore casing. The failure rate of water source from well or bores after an earthquake is approximately 76%.

In Chapter 4 it was shown that surface water (lakes, dams, reservoirs) were one of the most reliability sources for a water supply. This was due to the water typically being gravity fed to consumers in surface water, however, the length of pipes require to transport the water from the source of origin to the customers acts against the reliability of the supply in an earthquake. This results in surface water having a much higher probability of failure at approximately 45% when compared to on-site tanks (~13%). Both of these sources in normal operation (without an earthquake) exhibited a much closer level of reliability.

Once the adjusted source reliabilities are placed into the four scenarios from this report the reliability can be calculated for all the types of secondary water supply. The table in Appendix D shows the results of the post earthquake reliability of Classes A, B2 and C supplies under these conditions and Figure 23 and Figure 24 show the plots of these results.

Figure 23 Post Earthquake Availability Class C1 & C2

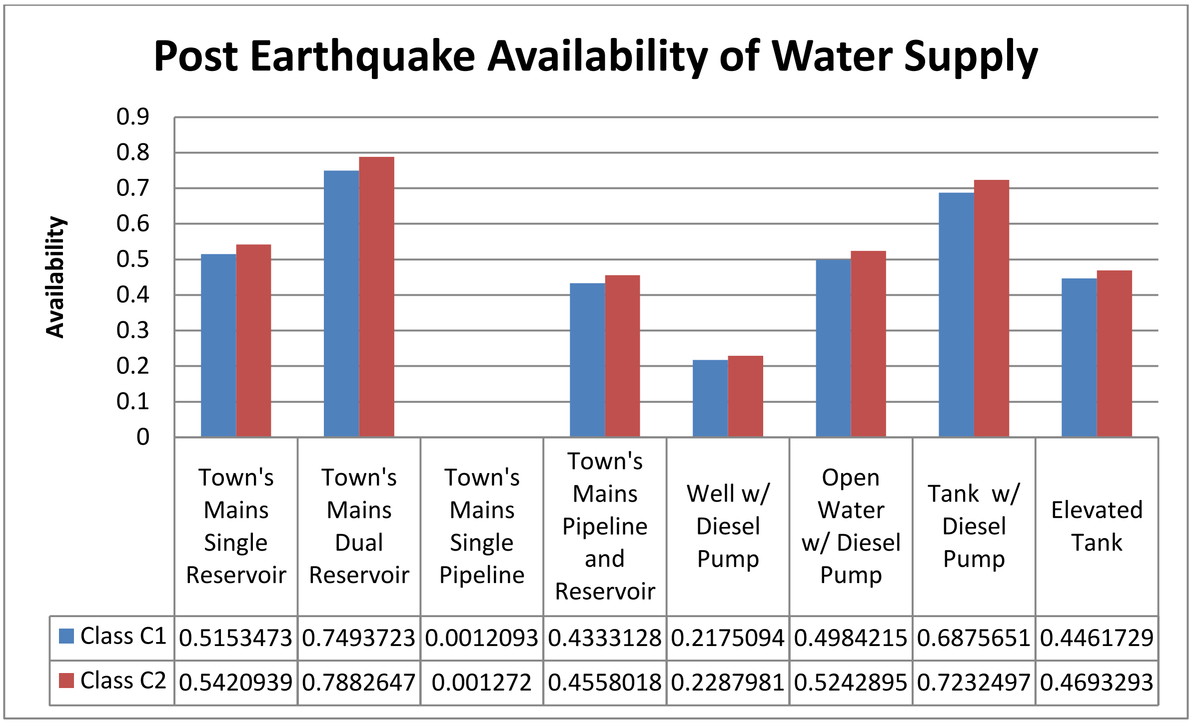
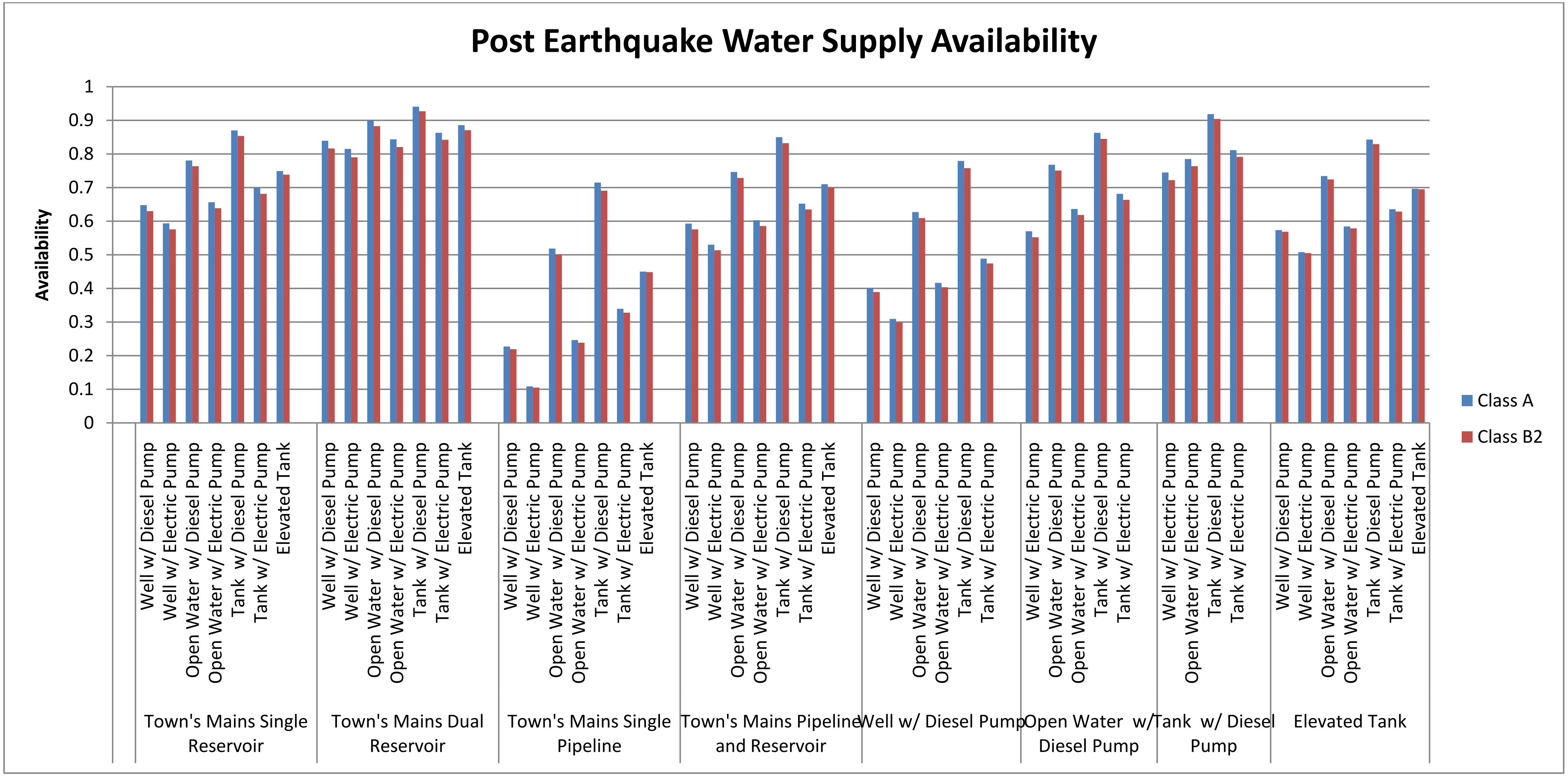


Figure 24 shows the relative influence of the local earthquake effects, the difference between a Class A and B2 supply, is much smaller than the effect that the choice of water supply source has on the availability of the supply. Further evidence of the importance of the selection of the supply can be seen in Figure 23, which shows a large range of reliabilities (almost 0% through to almost 79%) for water supplies after an earthquake. It also shows that a secondary diesel pump becomes more important after an earthquake with up to a 4% difference in the availability.

For a post earthquake scenario the results that Class A and B2 supplies are always more reliably than the Class C supplies no long holds. After an earthquake, the source of the primary supply becomes more important than the method of connection.

Figure 24 Post Earthquake Availability Classes A and B2



Chapter 7. Future Work

There are a number of areas in which future work would yield beneficial results.

- The costs of the equipment/infrastructure can be added to the model, this will allow for additional work into a cost-benefit analysis and the selection of the secondary water supply based on the reliability and the cost of installation.

- There is additional scope for research into the quality of the water supplied, with an assessment of the pressure variations due to the outages. This would establish the vulnerability of the supply network to reduced pressures. This also has an application in areas where the water pressure is marginal.

- The inputted data can be provided as distributions rather than a single value. This would allow for the inclusion of uncertainty in the results and would be beneficial in determining the level of error present in the final water supply figures.

- Inclusion of data directly from the network models would allow for increased accuracy for the drought and headworks reliability.

- Some of the data is collected in this report is for water supplies outside of New Zealand. It would be beneficial to change this data to specific failure rate taken from New Zealand equipment.

- There is some scope for the inclusion of more advance earthquake modelling. This would better account for the regional effects that the soil type, liquefaction and earthquake intensity would have on a water supply.

Chapter 8. Conclusions

In this report, a methodology for the calculation and assessment of the reliability of water supplies has been presented. This methodology consider four main scenarios for the supply of town water and a number of solution for locally water sourced from alternate supplies.

In order to assess the reliability of these supplies, critical components within the supplies need to be indentified. The reliability and impact of these components is then determined and a calculation of the availability of the supplies made.

The initial question that was asked in this report was “If the water supply is needed, what is the probability that water will be available to supply the sprinkler system?” It has been shown that in terms of the water supply conditions required by the New Zealand standard NZS 4541:2007 that the probability of failure ranges between 9.125×10^{-3} and 4.770×10^{-9}.

From these reliability it is possible to draw that following conclusion about the classes of supply:

- Class C2 supplies are always more reliability than a Class C1 of the same connection type.
- Classes A and B2 are always more reliable than either of the Class C connections regardless of the connection type.
- Class A supply will have a superior reliability when compared to a Class B2 of the same supply.

Therefore, the hypothesis drawn from the New Zealand standards that the reliability of a Class A is greater than Class B2 and in turn that is greater than Class C1 and Class C2 holds true for most cases found in this report.

The results for these supply have been shown to be similar to those found in Feeney when compared with the dual reservoir scenario. The reliability of the pumping infrastructure has be also been found to be similar to reliability reported in Crawley.

Earthquakes play and important factor in the selection of water supplies in New Zealand. The variability in the reliability of a water supply will be influenced by the effect these earthquakes have on the components.

This is evident in both the on-site and town’s main supplies with critical water supply functionality disrupted in all the cities investigated due to failures in the transmission and distribution pipelines. In the rare event of these earthquakes, it has been shown that the reduced pipe work required in a Class A supply has improved the ability of the water supply to continue functioning.

Earthquakes also have a considerably effect on the transmission and distribution of electricity. This is coupled with damage caused by ground movement to considerably alter the reliability of water supplies in a post-earthquake environment. This results in tank supplied water being the standout for reliability during an earthquake.

This research, as outlined in the scope, only applies to systems within New Zealand, and strictly speaking, the conclusions only apply to New Zealand water supplies. However, the similarity in the construction and maintenance of supply infrastructure and the requirements for secondary water supplies between New Zealand and many other countries could allow similar conclusion to be drawn. Provided that local condition are examined and entered into the model.

# Chapter 9.	References

Steel vs. Concrete – A life Cycle Cost Analysis. Philadelphia, Philadelphia Suburban Water Company.

(2002). Guidelines for Failure Impact Assessment of Water Dams. Brisbane Queensland Australia, The State of Queensland (Department of Natural Resources and Mines.

(2002). Guidelines for Failure Impact Assessment of Water Dams. Brisbane Australia, Department Of Natural Resources and Mines.

(2004). Environmental Assessment: Replace Potable Water Tanks and Provide Fire Suppression at Park Headquarter Sequoia & Kings Canyon National Parks California, National Park Service U.S. Department of the Interior.

(2006). Rainwater Tank Modelling Investigation: South East Queensland Regional Water Supply Strategy. SPRING HILL QLD 4004 Australia, WBM Oceanics Australia.

(2007). Planning Strategy for Metropolitan Adelaide. Building South Australia. Adelaide, Government of South Australia.

Adams, J. (2004). Implications of Canada's 4th Generation Seismic Hazard Model for Canadian Dams. Ottawa, Canada., Canadian Dam Association

Adams J. and Lin L. (2008). Seismic vulnerability and prioritization ranking of dams in Canada. The 14th World Conference on Earthquake Engineering, , Beijing, China.

Al-Fozan, S. A., M. Al-Hajri, et al. (2004). Investigations Of The Corrosion Of Water Storage Tank In Al-Khobar Plant. Al-Jubail, Saudi Arabia, Research and Development Centre Saline Water Conversion Corporation.

Allen, D. (1998). "Web-Based Process Engineering Risk Calculator." Retrieved 12th December, 2009, from http://www.cheque.uq.edu.au/ugrad/theses/1998/DaveA/help.html.

Australian Government National Water Commission (2007). 2006-2007: National Performance Report, Urban Water Utilities Australian Government.
Avi Ostfeld, Dimitri Kogan, et al. (2002). "Reliability simulation of water distribution systems – single and multiquality." UrbanWater 4: 53-61.

Balbus, J., T. Graff, et al. (2004). Solutions For Restoring Yosemite's Hetch Hetchy Valley, Environmental Defence.

Ballantyne, D. The Shakeout Scenario Supplemental study. Denver Colorado USA, SPA Risk LLC,.

Ballantyne, P. E., Crouse, C.B., (1997). Reliability and Restoration of Water Supply Systems for Fire Suppression and Drinking Following Earthquakes. Gaithersburg, MD, National Institute of Standards and Technology.
Basak, N. N. (2003). Environmental engineering. Noida India Tata McGraw-Hill.

Al-Barqawi, H and Zayed, T (2006) Assessment Model of Water Main Conditions, ASCE Conf. Proc. 211, 27 (2006), DOI:10.1061/40854(211)27

Bedford, T., van Gelder, P.H.A.J.M (ed.) (2003). Safety & Reliability - Lisse, the Netherlands, A.A. Balkema.

Bennetts, l, Poh, S.L.,Thomas, I.R Lee A.C, Beever, P.F., Ramsay, G.C and Timms, G.R., (1998). Fire Safety In Shopping Centres. F. R. R. P. 6. Sydney, Australia, Fire Code Reform Centre LTD.

Bennetts, I.D., Poon, S.L., Poh,K.W., (1996) Fire Safety Design, Proceedings Design of Steel Building for Fire Safety, Australian Institute of Steel Construction Sydney, Australia.

Botting R. and A. H. Buchanan (1998). Structural Design for Fire After Earthquake. Australasian Structural Engineering Conference, Auckland, New Zealand Structural Engineering Society.

Cadwallader L.C. (1995). Fire Protection system Operation Experience Review for Fusion Applications. Idaho Falls, Idaho, Idaho National Engineering Laboratory Nuclear Engineering Technologies Department Lockheed Technologies Company.

Coombes, P. J. "Integrated Water Cycle Management: Analysis of Resource Security." Water Vol. 32, Issue 2, p. 21-26

Coombes, P. J. (2008). Rainwater Tanks Revisited: new opportunities for urban water cycle management Newcastle Australia, University of Newcastle.

Cousins, W. J., Spense, R.J.S., So, E., (2008). Estimated casualties in New Zealand earthquakes. Conference Australian Earthquake Engineering Society. Ballarat

Crawley, P. (1993). Reliability assessment of Components of the Metroplitan Adelaide Headworks Pumping System. Adelaide, Australia, Operations Support Branch Engineering and Water Supply Department.

Crawley, P. (1995). Risk and Reliability Assessment of Multiple Reservoir Water Supply Headworks Systems. Faculty of Engineering. Adelaide, University of Adelaide. Doctor of Philosophy.

Damelin E., Shamir U., and Arad N. (1972). "Engineering and Economic Evaluation of the Reliability of Water Supply." Water Resources Research 8(4): 861-877.

Daniela E. Popescu, C. P., Gianina Gabor (2003). Monte Carlo Simulation using Excel for Predicting Reliability of a Geothermal Plant. International Geothermal Conference,. Reykjavík.

Diment,G.A. (1991), The Wide Use of a General Headworks and Resources Simulation Model – REALM, “Real-World Applications of Optimisation Techniques to Water Resources Management”, International Hydrology and Water Resources Symposium 1991, The Institute of Engineers, Australia, Perth W.A , Australia, 1991

Donnelly, C. R. (2006). Safe and secure - risk-based techniques for dam safety. International Water Power and Dam Construction

Energex (2009). Annual Report -2008. Brisbane.

Falconer, R. Earthquake Risk Assessment Methods for Energy Supply Systems. Lower Hutt, New Zealand, Institute of Geological & Nuclear Sciences Limited.

Feeney, M. (2001). Accounting for Sprinkler Effectiveness in Performance Based Design of Steel Buildings for Fire. Department of Civil and Natural Resources Engineering. Christchurch, Univeristy of Canterbury.

Flanagan, K. (2007). Status Report - Drought Contingency Plan. Toowoomba.

Gibbons, A. (2009). Annual Reliability & Power Quality Report Financial Year Ending June 2009, Western Power.

Greater Wellington Council. (2004). "Greater Wellington Water Asset Management Plan." Retrieved 5th of January, 2010, from http://www.gw.govt.nz/assets/council-publications/Asset%20Management%20Plan%202004.pdf.

Hydraulic Institute, Europump, and the US Department of Energy's Office of Industrial Technologies (OIT). "Pump Life Cycle Costs:A Guide To Lcc Analysis For Pumping Systems" Retrieved 10th January, 2010, from http://www.fluidcomponents.com/Aerospace/Products/PressureSensors/A_ProdPressure_ASPS.asp

Hydro Review (1999). "Dam Safety." Vol XVIII(No. 5): 84-85.

Knoy, E. C. (1995). Performance of Elevated Tanks During Recent California Seismic Events. AWWA Annual Conference & Exhibition.

Kuczera, G. (1991) Use of Generalized Headworks Simulations Models as Planning Tools, Real-World Applications of Optimisation Techniques to Water Resources Management", International Hydrology and Water Resources Symposium 1991, The Institute of Engineers, Australia, Perth W.A , Australia, 1991

Ko, G. P.-H. and C.-S. Yang (1999). Introduction to Reliability Analysis in Dam. Department of Mechanical Engineering. Hsinchu Taiwan, National Chiao-Tung University.

lan R. Thomas and l. Bennetts (2003). "The Effect of Fire in the Office Building at 140 William Street." American Institute of Steel Construction.

Mannam, S. (2005). Lee's Loss Prevention in the Process - Industries Hazard Identification and Control 3rd edition. MA, USA, Elsevier Butterworth-Heinemenn

Marsden Jacob Associates (2007). The cost-effectiveness of rainwater tanks in urban Australia, National Water Commission.

McElhaney, K. L. and R. H. Staunton (1996). Reliability Estimation from check Valves and Other Components. ASME Pressure Vessel &Piping Conference. Montreal, Canada.

Miller, R., J. Guice, et al. (2008). Risk Assessment for Drinking Water Sources, Water Corporation and Cooperative Research Centre for Water Quality and Treatment.

Ministry of Health (2005) " Drinking Water Standards for New Zealand 2005", Wellington NZ, Ministry of Health.

Nadebaum, P., M. Chapman, Morden R, Rizak S.. (2004). A Guide To Hazard Identification & Risk Assessment For Drinking Water Supplies, CRC for Water Quality and Treatment.

National Weather Service (2008) " Medical Aspects of Lightning" Retrieved 5th of January, 2010, from http://www.lightningsafety.noaa.gov/medical.htm

New South Wales Health, (2010) "NSW Health Response Protocol: following failure in water treatment or detection of giardia or cryptosporidium in drinking water" Retreived 28th of April ,2010, from http://www.health.nsw.gov.au/publichealth/environment/water/response.asp

Nisar A., Scawthorn C., Stillman C., Jasperse J.,Gur T.,Villet W.. (2006). Multi-hazard reliability for a major water utility agency in the San Francisco bay area. .

The 8th U.S. National Conference on Earthquake Engineering, San Francisco, California, USA.

Orion Group (2009). Network Quality Report.

O'Rourke, M. and Bouabid, J., (1996). Analytical Damage Estimates for Concrete Pipelines. 11th World Conference on Earthquake Engineering. Acapulco, Mexico.: paper no. 346.

Davenport P.N., Lukovic B., Cousins.W.J.. (2006). Restoration of earthquake damaged water distribution systems. NZSEE Conference.
Pan American Health Organization. (1998). "Annex 3 - Method for Estimating Damage in Pipes as a Consequence of Intense Earthquakes."   Retrieved 10th August, 2009.

Pim, J. H., Searing J.M. (1988). Tank corrosion study. Farmingville, Long Island, New York, Suffolk County Department of Health

PriceWaterhouseCoopers (2007). National Performance Report 2006–07 Part A, Water Services Association of Australia, the National Water Commission and the National Water Initiative Parties.

PriceWaterhouseCoopers (2007). South East Queensland Water Sector Asset Audit, Queensland Water Commission.

Proe.D. J, and Bennets, I.D (1994). Real Fire Test in 380 Collins Street Office Enclosure, BHP Melbourne Research Laboratories.

Proe.D.J, and Bennets, I.D (1997). Six Fire Tests in a Large Sprinklered-Protected Open Plan Office, BHP Melbourne Research Laboratories.

Queensland - Fire and Rescue Service. (2005). "Australian Standard 1851-2005 “Maintenance of Fire Protection Systems and Equipment”." Retrieved 5/11/09, 2009.

Risk & Reliability Associates (2009). Critical Assets Due Diligence Review, Gladstone Area Water Board

Rotation Moulders Australasia Inc. (2009). "Buying & Maintaining Your Poly Water Tank." Retrieved 4th of December, 2009, from http://www.watertanks.org.au/downloads/Tank_Guide.pdf.

SA Water. "Education - Our Water System." Retrieved 10th August, 2009, from http://www.sawater.com.au/SAWater/Education/OurWaterSystems/Filtration.htm.

SEQwater. “Managing Our Dams and Weirs” Retrieved 10th August, 2009, from http://www.seqwater.com.au/public/catch-store-treat/dams

Scott Gould, Fanny Boulaire, et al. Understanding How the Australian Climate Can Affect Pipe Failure. Melbourne, Australia, CSIRO and Monash University.

Shamir, U. and C. D. D. Howard (1981). "Water supply reliability theory." AWWA.

Shamsi, U. M. (1990). "Computerized Evaluation of Water - Supply Reliability." IEEE TRANSACTIONS ON RELIABILITY, 39(1): 35-41.

Shinozuka, M. (1991). "Lifeline Seismic Disaster Mitigation : Research and Implementation." Retrieved 20th January, 2010.

Smith, R. J. (2007). WASTE WATER INDUSTRY PUMP RELIABILITY, AESSEAL plc.

South Australian Government, Atlas of South Australia. Retrieved 12th January, 2010 from http://www.atlas.sa.gov.au/

Standards Australia (1999). Automatic Fire Sprinklers Systems AS2118:1999. Canberra ACT Australia, Standards Australia.

Standards Australia (2006). Automatic Fire Sprinklers Systems AS2118:2006. Canberra ACT Australia, Standards Australia.

Standards New Zealand (2007). NZS:4541 - Automatic Fire Sprinkler Systems. Wellington, Standards New Zealand.

Stefano Alvisi, Marco Franchini. (2008). "Comparative analysis of two probabilistic pipe breakage models applied to a real water distribution system " Civil Engineering and Environmental Systems Volume 27(Issue 1 March 2010): pages 1 - 22

Strongman, B. (2004). Reliability: Beyond the numbers, Using Key Performance Measures. TECHBrief., Burns& McDonnell

Sydney Catchment Authority (2007). Review of Sydney's Water Supply System Yield. Sydney Australia, Sydney Catchment Authority.

The Cooperative Research Centre (CRC) for Water Quality and Treatment (2008). Drinking Water Treatment. Adelaide Australia.

The Cooperative Research Centre for Water Quality and Treatment (2008) "All about Australia's drinking water."

Thomas, I. R., l. Bennetts, Dayawansa, P.,Proe. D.J., Lewins, R. R. (1992). Fire Test of the 140 Williams Street Office Building. Melbourne Australia, BHP research.

Tolson, B. A., H. R. Maier, Simpson A.R., Lence B.J.. (2004). "Genetic Algorithms for Reliability-Based Optimization of Water Distribution Systems." Journal of Water Resources Planning and Management.

Victoria University and Department Of Sustainability and Environment (2005). REALM User Manual. Melbourne Australia.

Wagner, J. M., U. Shamir, Marks. D.H., (1988). "Water Distribution Reliability: Simulation Methods." Journal of Water Resources Planning and Management, 114(3): 276-294.

Water Services Association of Australia, the National Water Commission, and the NWI Parties (2007). National Performance Report 2006-2007 urban water utilities Part B.

Water Services Association of Australia, the National Water Commission and the NWI Parties. (2007). National Performance Report Urban 2006-2007 Part C, Water Services Association of Australia Ltd.

Wei-yun, Z. T.. (1999). "Numerical analysis of buried pipe characteristics." Journal of Zhejiang University - Science A Volume 1(Number 2 / April, 2000): 144-147.

Appendix A. Extract from New Zealand Sprinkler Standards – for commercial buildings - NZS 4541:2007

P a r t 6

WATER SUPPLY

601 GENERAL

To ensure its reliability, every sprinkler system shall have a reliable primary water supply complying with this section. An additional or secondary water supply is required by this Standard where the building presents a greater risk to occupants or contents and is larger or higher, thus being more difficult for the Fire Service to control a fire (see 602.5).

A secondary water supply shall not be required for a sprinkler system to comply with the Compliance Documents of the New Zealand Building Code.

602 CLASSES OF WATER SUPPLY

602.1 Classification

The water supply of a sprinkler system shall be classified according to its reliability and number of supplies as follows:

Class A – Dual superior supply
Two approved supplies, both of which shall be carried independently to a combined main within each control valve enclosure, at least one of which shall be a primary supply, but only one of which may be dependent on a town's main.

Class B2 – Private site fire main
A private reticulation system which complies with 612 and is reserved solely for fire purposes, being charged with water and normally pressurised, comprising of at least one approved ring and supplied by two approved supplies, at least one of which shall be a primary supply, but only one of which may be dependent on a town's main. The sprinkler system(s) water supply connection pipes shall be independently connected to the fire main. Two connections shall be carried

independently to a combined main within each valve enclosure. Isolation valves shall be provided at each connection with the fire main so as to ensure that there are at least two isolation valves between any two connections.

Class C2 – Single superior supply
A single approved primary supply. Where the supply is reliant on the use of fire pumps, two pumps shall be provided in parallel. Each pump shall be individually capable of meeting the highest design flow and pressure. At least one pump shall be driven by a diesel engine.

Class C1– Single supply
One approved primary supply.
NOTE – See figure 6.1.

602.2 Primary supply

A primary supply shall be only one of the following:
(a) A town's main, boosted town's main, or supplemented town's main provided that any pump is driven by a diesel engine;
(b) A diesel engine driven pump taking water from an approved source other than a town's main;
(c) An elevated tank.

602.3 Secondary supply

A secondary supply shall be one of the following:
(a) A town's main, boosted town's main, or supplemented town's main where any pump is either diesel engine or electric motor driven;
(b) A diesel engine or electric motor driven pump taking water from an approved source other than a town's main;
(c) An elevated tank.

602.4

Every water supply shall:

(a) Automatically provide water at least at the design flows and pressures specified in 603;

(b) In cases other than town's mains, boosted town's mains or supplemented town's mains, have a storage capacity of at least that specified in 606;

(c) Meet the requirements of 604, 605, 606, 607, 608 or 609 as they relate to the particular water supply (see figure 6.2);

(d) Except in the case of town's mains, be under the direct control of the owner of the protected building. Where this is not practical, approval may be given to a legally binding arrangement which suitably guarantees the right of use of the building owner to the water supply.

NOTE –

(1) The use of salt or brackish water is not normally allowed. In special circumstances where there is no suitable fresh water source available, consideration may be given to the use of salt or brackish water, provided the installation is normally charged with fresh water.

(2) Attention is drawn to the need to comply with the requirements of the New Zealand Building Code Clause G12.

602.5 Minimum water supply requirements

602.5.1 Class A and B2 water supplies

A Class A or Class B2 water supply as defined in 602.1 is required for the following buildings:

a) Buildings greater than 25 m high, measured from the point of lowest entry to the floor level of the highest normally occupied floor, and located in areas having a hazard factor *Z*, greater than 0.13 (as defined in NZS 1170.5). For further information, see Appendix H;

b) Buildings or groups of buildings used for crowd or sleeping occupancies, with a total floor area of more than 11,000 m2, unless the building is subdivided into sprinkler protected floor areas of 11,000 m2 or less by one of the following:
    (i) Walls having a FRR of 60/60/60
    (ii) External walls at least 10 m from the external walls of other buildings
    (iii) Combinations of (i) and (ii).

602.5.2 Class C2 water supply

A Class C2 water supply, as defined in 602.1 is the minimum required for buildings greater than 25 m high, measured from the point of the lowest entry to the floor level of the highest normally occupied floor, and located in areas having a hazard factor Z, equal to or less than 0.13 (as defined in NZS 1170.5). For further information, see Appendix H.

602.5.3 Class C1 water supply

A Class C1 water supply as defined in 602.1 is the minimum requirement for all buildings not described in 602.5.1 or 602.5.2.
NOTE :A Class C1 water supply is the only supply required for a sprinkler system to comply with the Compliance Documents of the New Zealand Building Code.

605.1 Acceptable sources

The following are acceptable sources of water for pump units, provided they satisfy the detailed requirements set out in this Standard:

a. Town's mains (see 604);
b. Tanks (see 606.1 and 606.3);
c. Wells and artesian bores (see 606.4);
d. Open water (see 606.5).

In each case, the water shall be clean and free from sediment and debris.

Appendix B Australia Standard

The Building Code of Australia still only reference the 1999 version of the Automatic fire sprinkler systems code. With a statement on the title page of the 2006 edition reading: *'This edition of AS 2118.1 is not referenced in the BCA. The 1999 edition, which this edition updates, continues to be referenced in the BCA. Both editions will achieve the same objective, however, this edition is predicated on advanced technology resulting in a more cost-effective sprinkler design and may be used as an alternative solution.'* 2118:1999 provides for grades of water supplies that need to be met for each of the hazard groups.

Extract from the 2006 Australian Standards for Automatic fire sprinkler systems:

4.2 DUAL WATER SUPPLIES

4.2.1 General

Dual water supplies shall be provided where

a) a building is defined as 'high-rise', or
b) required by building owners, insurers or fire engineers for High Hazard occupancies.

and shall comprise any two of the following:

i) Town main (see Clause 4.3.2).
ii) Town main with automatic booster pumps (see Clause 4.3.2.2).
iii) A private system water supply (see Clause 4.3.3).
iv) A private system water supply with automatic booster pumps (see Clause 4.3.3.1).
v) Suction tank with automatic pumps (see Clause 4.3.4).
vi) Natural sources such as rivers, lakes or underground water supply, subject to the conditions set out in Clause 4.3.5, with automatic pumps.
vii) Gravity tank (see Clause 4.3.6).
viii) Elevated private reservoir (see Clause 4.3.7).
ix) Pressure tank–permissible for Light and Ordinary Hazard 1 classes only (see Clause 4.3.8).

C4.2 Dual water supplies may be mandated by regulatory authorities for sprinkler systems serving high-rise buildings (buildings exceeding 25m effective height) where they are generally considered beyond the reach of fire brigade aerial appliances.
However; dual water supplies are also frequently specified by insurers and fire engineers for risks involving substantial loss exposure and/or potential environmental hazard.

4.2.2 Acceptable arrangements

4.2.2.1 Independent arrangement

Dual water supplies shall be independent, or form part of a gridded town main system, and shall have stop valves so arranged that in the event of a breakdown at least one supply remains operative.

4.2.2.2 Individual connections

Where dual water supplies consist of two individual connections from a town main system, each connection shall be carried separately to inside the building structure, as follows:

a) Where booster pumps are installed, each connection from the town main system shall be carried separately to the pump suction manifold described in Clause 4.3.9.3. An isolating valve shall be installed upstream of the pump suction manifold on each connection from the town main system. Each of these two isolating valves shall be secured open and shall be positioned as close as practicable to the isolating valves provided, in accordance with the requirements of Clause 4.3.9.3 at each pump inlet (see Figure 4.2.2.2(A)).

b) Where booster pumps are not installed, each connection from the town main system shall be carried separately to a point as close as practicable to the sprinkler-protected building, where they may be interconnected. A single (combined main) connection may be carried from the interconnection point into the building (see Figure 4.2.2.2(B)).

C4.2.2.2(b) The isolating valve on each of the two connections from the town main system, required upstream and in close proximity to the pump suction manifold, provide a means of closing off any damaged or non-operational town main connection, thus preventing any operating pump from drawing air (see also Clause 4.3.9.3).

4.2.2.3 Pumps

Where each supply requires a pump, one pump shall be compression ignition engine-driven, the other may be electric motor-driven.

4.2.2.4 Tanks

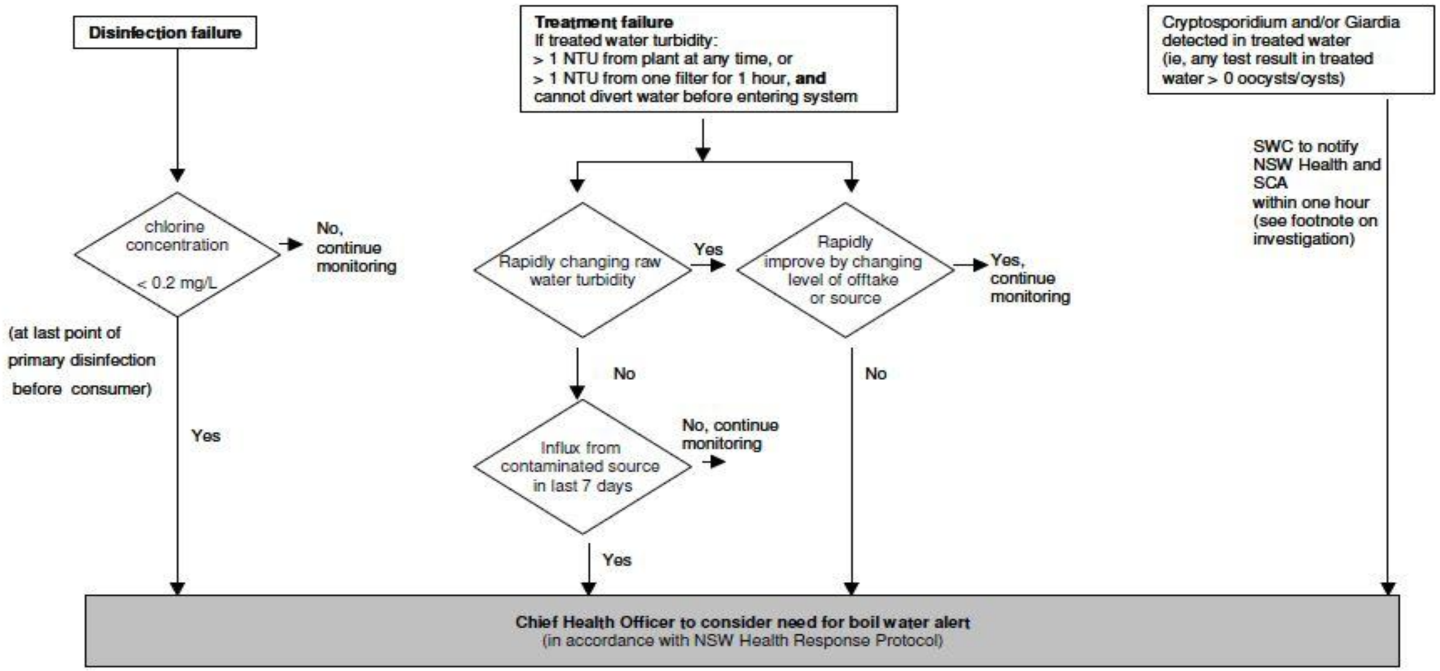
Where both supplies are comprised of tanks, with or without pumps, the tanks shall be either separate units or a single unit partitioned to hold the required water storage capacity in each compartment.

Appendix C Water Treatment Flow Chart

Figure 25 Flow Chart of Treatment Plant Failure for Sydney Water – NSW Health 2010

Response Protocol flow chart – Sydney water supply

Response Protocol following failure in water treatment or detection of Giardia or Cryptosporidium



The investigation triggered by the detection of Cryptosporidium and/or Giardia should include an assessment of: disinfection, treatment and treated water turbidity (as shown in the flow chart), accuracy of findings (confirmation, recovery rate, Independent Laboratory Result), the presence of indicator bacteria, raw water turbidity, pre-treatment contamination of raw water, particle counts where available, and post- treatment contamination.

Resampling should aim to determine the extent of the contamination.

Appendix D Earthquake Model

Table 34 Earthquake model survival rates

Description	Survival Rate
Underground Water Sources	
Aquifer stops Producing	0.9
Sanding of the well/bore	0.85
Structural Failure	0.8
Non-Structural failure	0.55
Indirect Damage	0.99
Well/Bore Casing Damage	0.99
Slab and Casing Premanently Displaced	0.95
Loss of Main's Power	0.05
Loss of Backup Power	0.75
Aquifer Contaminated	1
Flooding to the Bore/Well Head	0.99
Damage to the Bore/Well Head	0.99
Transfer pipe from Well/Bore to Treatment Plant	0.9
Pumping Failure	
Structural Failure	0.8
Non- Structural Failure	0.25
Indirect Damage	0.95
Loss of Main's Power	0.05
Loss of Backup Power	0.45
Tanks	
Pipe Complete Failure	0.9625
Tank Structural Damage	0.95
Inlet/ Outlet Failure	0.95
Pipe	
Isolating Pipe Failure	0.95
Excessive Pipe Failure	0.15
Non Isolating Pipe Failure	0.95

Surface Water	
Source Contamination	1
Intake Blocked	0.85
Structural Failure	0.99
Non_structural Failure	0.65

Figure 26 Fault Tree for Pipe Failure

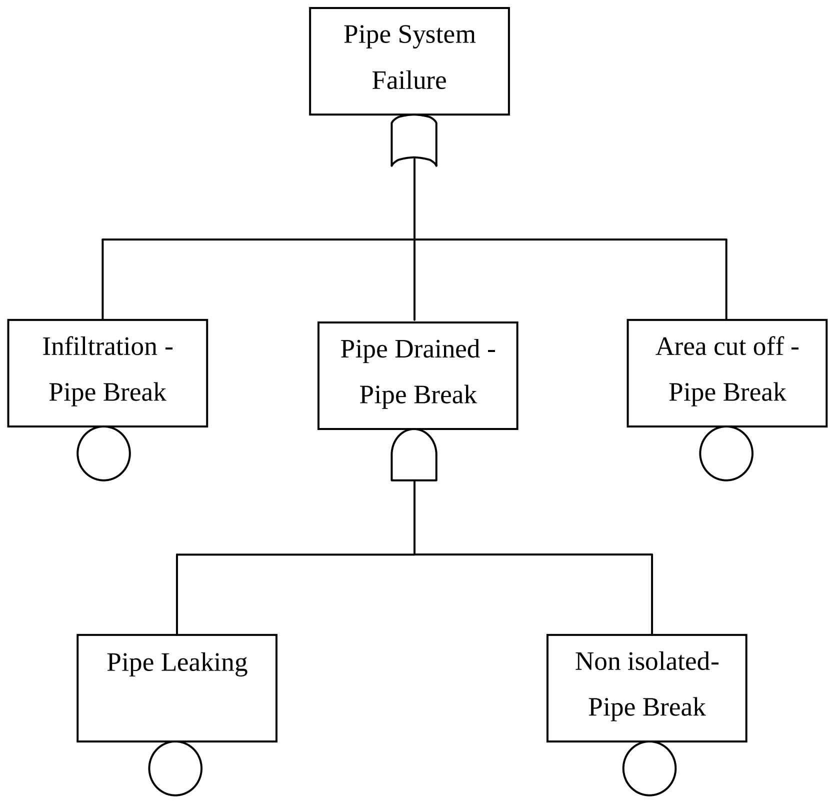


Figure 27 Fault Tree for Tanks

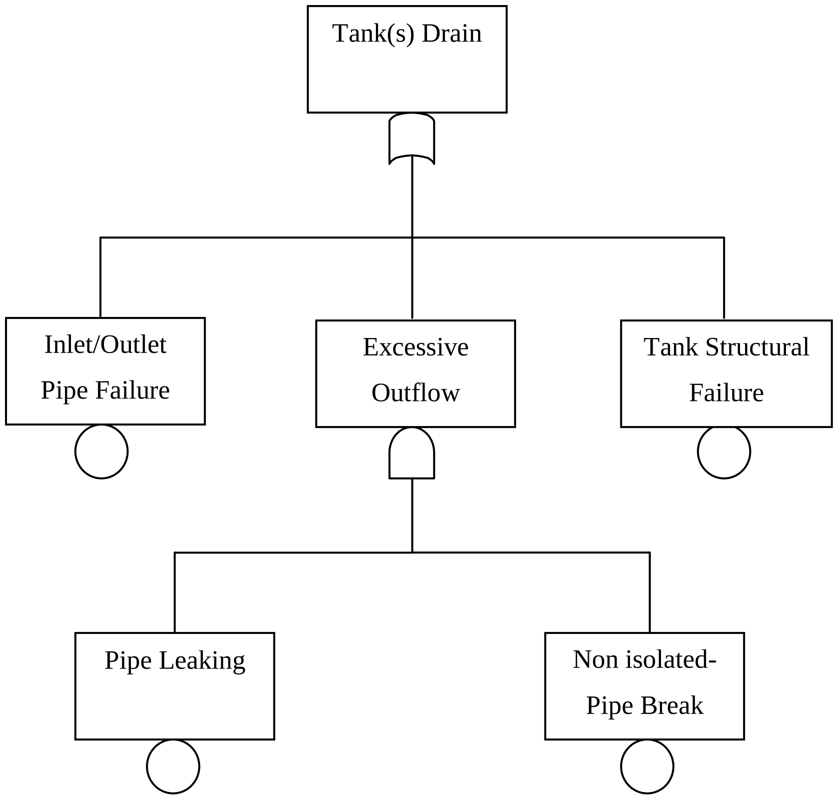


Figure 28 Fault Tree for Inadequate Water

Pipe System Failure, and Tank(s) Drain expanded in previous fault trees. Source Water Fails fault tree appears on page 128.

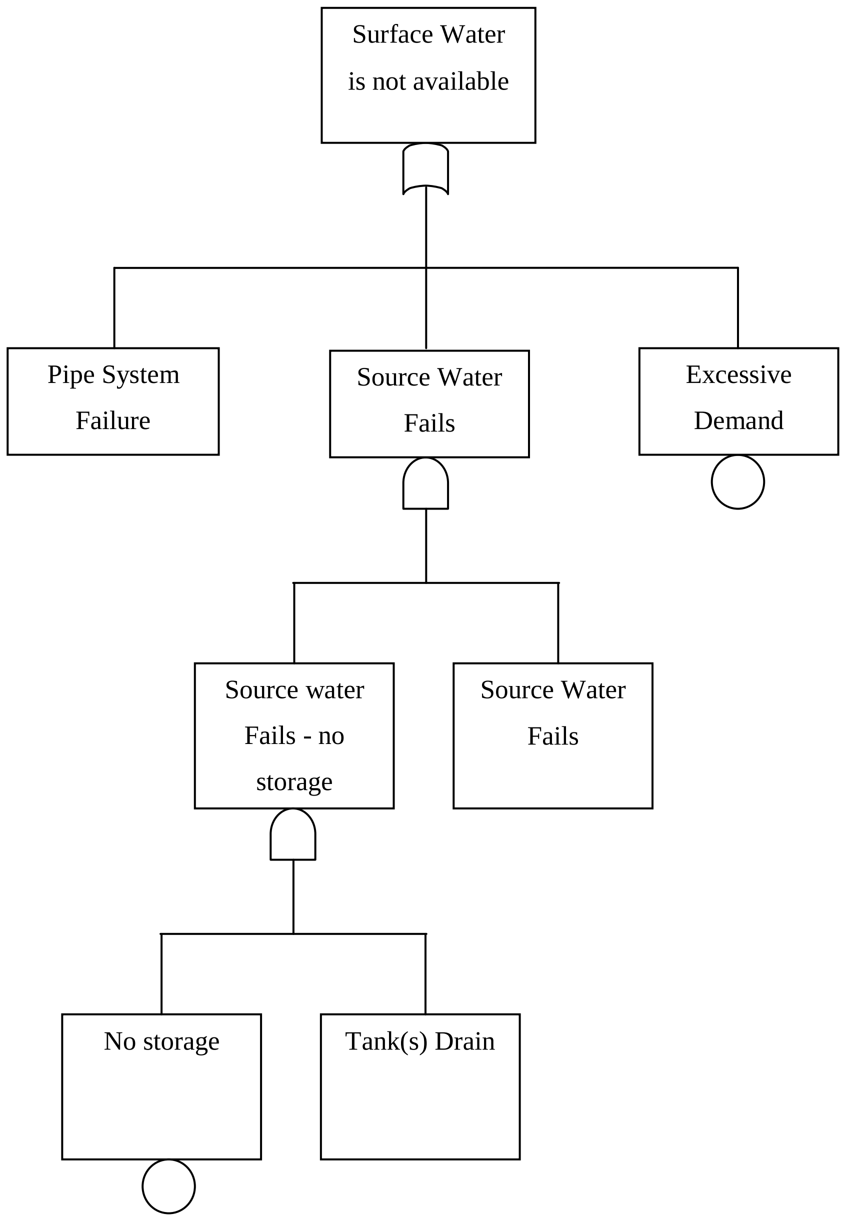


Figure 29 Fault Tree for Pumps

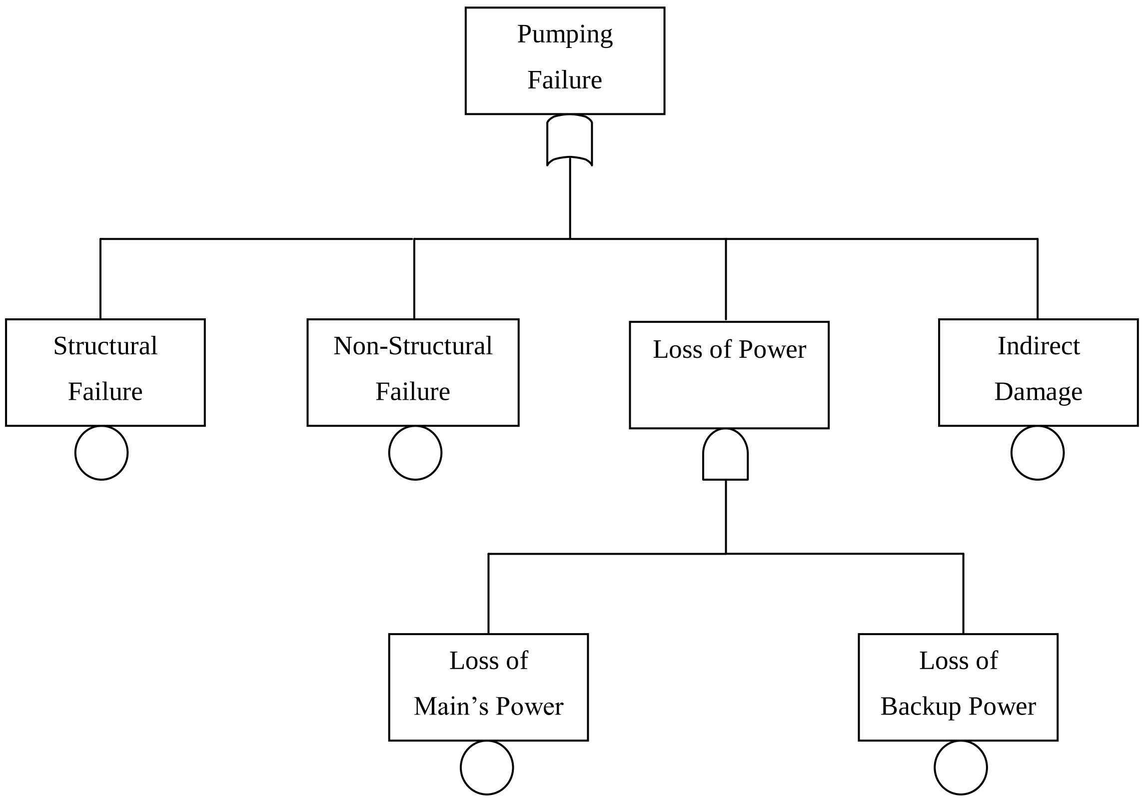


Figure 30 Fault Tree for Well/Bore Failure

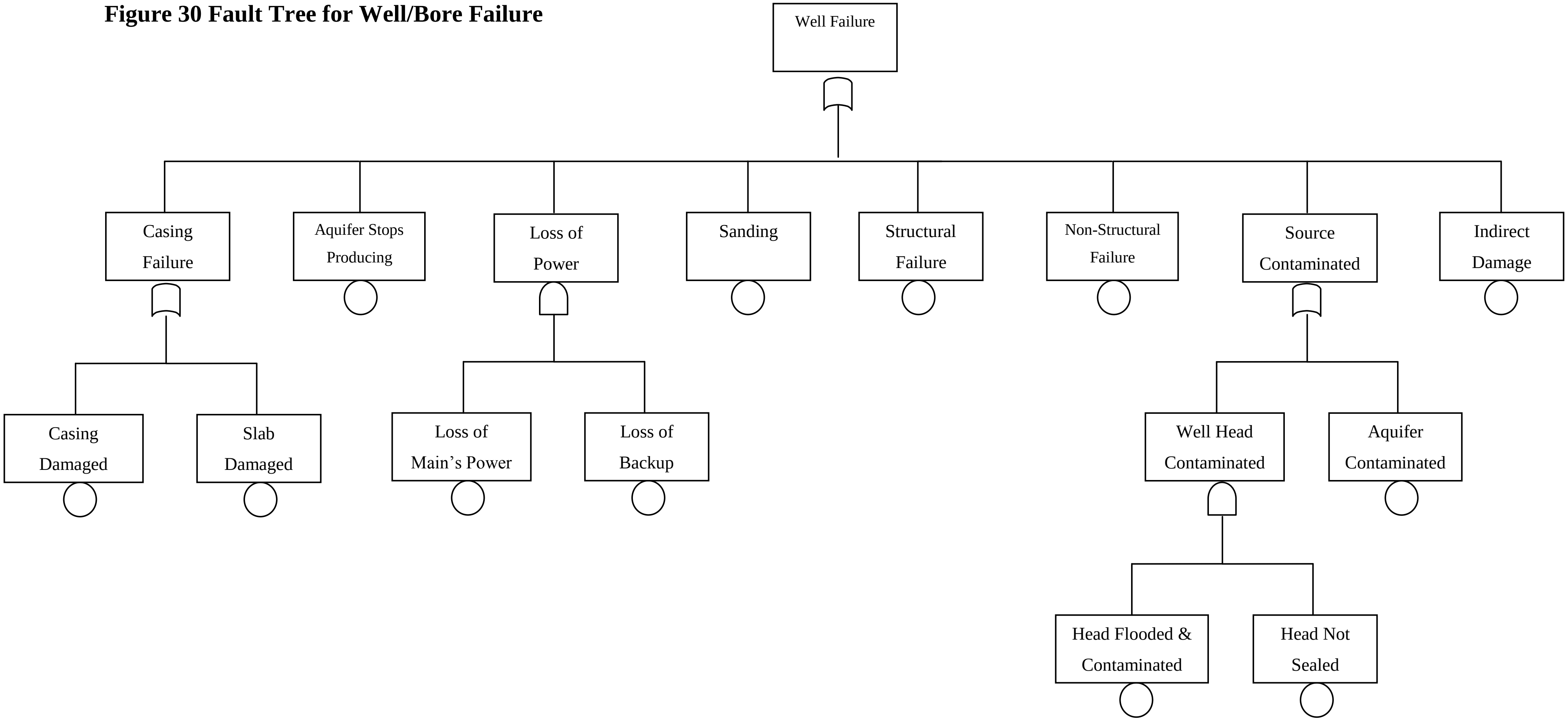


Figure 31 Fault Tree for Surface Water

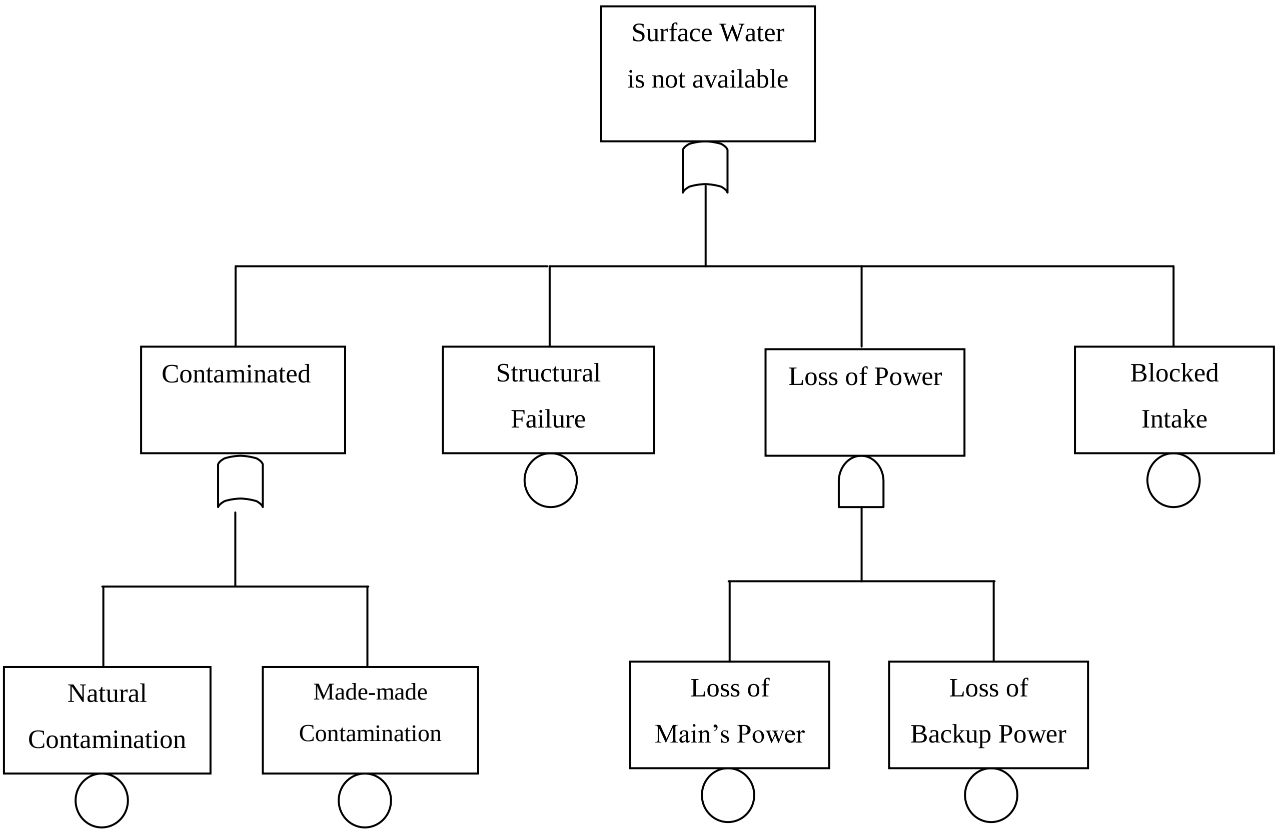
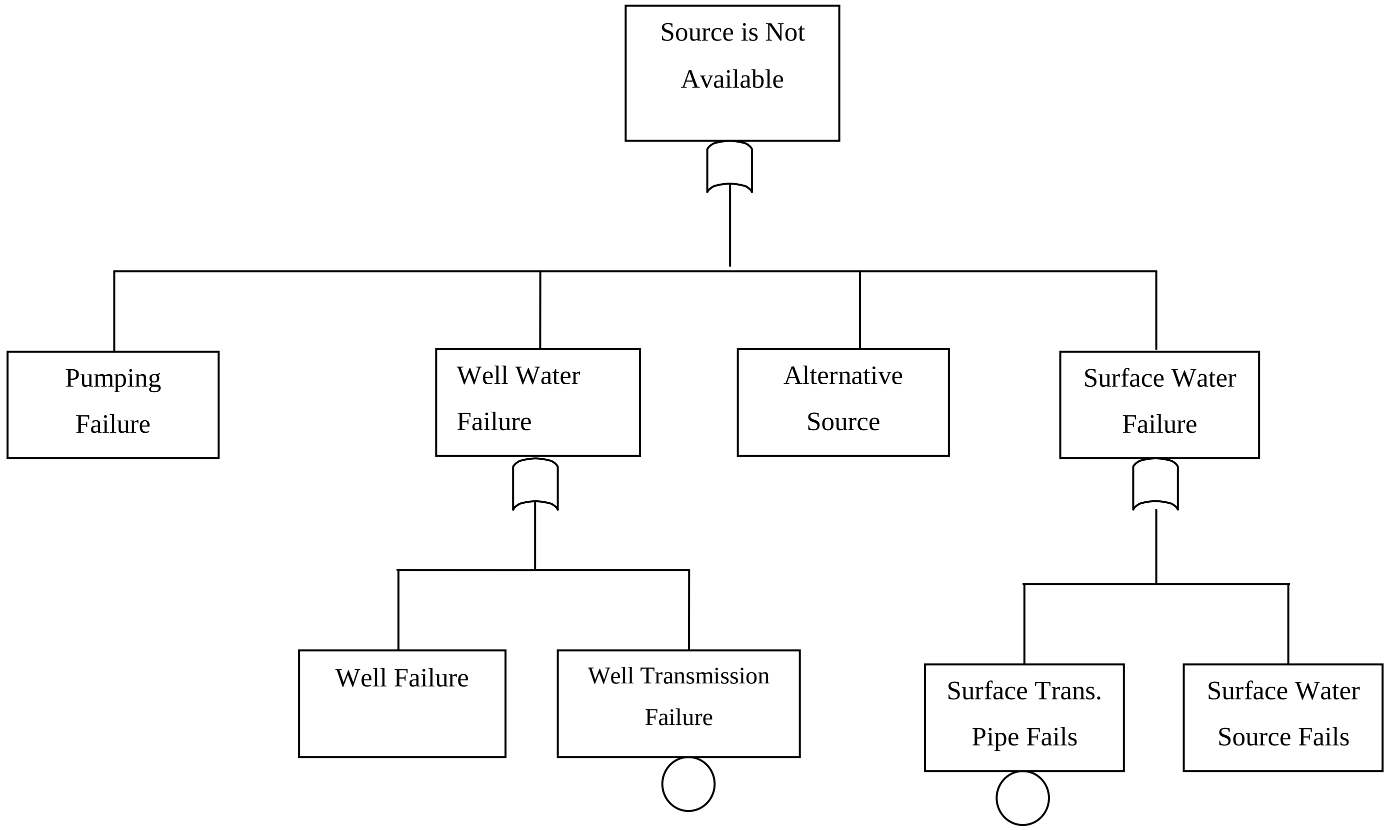


Figure 32 Fault Tree for Source Water

Pumping Failure, Well Failure, and Surface Failure expanded in previous fault trees.



Appendix D2 Availability if Earthquake occurs.

Table 35 Availability of supplies after an earthquake

Primary	Secondary	Class A	Class B2	Class C1	Class C2
Town's Mains Single Reservoir	Well w/ Diesel Pump	0.9996578689	0.9982455906	0.9960365753	0.9980286485
	Well w/ Electric Pump	0.9996583023	0.9982477606		
	Open Water w/ Diesel Pump	0.9997386362	0.9986500207		
	Open Water w/ Electric Pump	0.9997391019	0.9986523528		
	Tank w/ Diesel Pump	0.9999970701	0.9999440906		
	Tank w/ Electric Pump	0.9999976395	0.9999469414		
	Elevated Tank	0.9999999167	0.9999953108		
Town's Mains Dual Reservoir	Well w/ Diesel Pump	0.9999823499	0.9985740700	0.997383083	0.999377849
	Well w/ Electric Pump	0.9999999957	0.9985758337		
	Open Water w/ Diesel Pump	0.9999865166	0.9989027783		
	Open Water w/ Electric Pump	0.9999865406	0.9989046738		
	Tank w/ Diesel Pump	0.9999998489	0.9999545586		
	Tank w/ Electric Pump	0.9999998782	0.9999568756		
	Elevated Tank	0.9999999957	0.9999961888		

Primary	Secondary	Class A	Class B2	Class C1	Class C2
Town's Mains Single Pipeline	Well w/ Diesel Pump	0.9978311609	0.9963963733	0.9884562365	0.9904331490
	Well w/ Electric Pump	0.9978339081	0.9964008306		
	Open Water w/ Diesel Pump	0.9983431611	0.9972270888		
	Open Water w/ Electric Pump	0.9983461135	0.9972318790		
	Tank w/ Diesel Pump	0.9999814270	0.9998851600		
	Tank w/ Electric Pump	0.9999850360	0.9998910155		
	Elevated Tank	0.9999994721	0.9999903682		
Town's Mains Pipeline and Reservoir	Well w/ Diesel Pump	0.9998876496	0.9984782027	0.9917911770	0.9937747593
	Well w/ Electric Pump	0.9998877919	0.9984800849		
	Open Water w/ Diesel Pump	0.9999141723	0.9988290105		
	Open Water w/ Electric Pump	0.9999143252	0.9988310334		
	Tank w/ Diesel Pump	0.9999990379	0.9999515035		
	Tank w/ Electric Pump	0.9999992248	0.9999539763		
	Elevated Tank	0.9999999727	0.9999959325		

Primary	Secondary	Class A	Class B2	Class C1	Class C2
Well w/ Diesel Pump	Well w/ Diesel Pump	0.9422238229	0.9401039503	0.7548190813	0.7563287195
	Well w/ Electric Pump	0.9422970055	0.9401780347		
	Open Water w/ Diesel Pump	0.9558631075	0.9539113121		
	Open Water w/ Electric Pump	0.9559417568	0.9539909305		
	Tank w/ Diesel Pump	0.9995052310	0.9980912381		
	Tank w/ Electric Pump	0.9996013721	0.9981885639		
Open Water w/ Diesel Pump	Well w/ Electric Pump	0.9559190138	0.9539683184	0.8112031096	0.8128255158
	Open Water w/ Diesel Pump	0.9662825515	0.9645357722		
	Open Water w/ Electric Pump	0.9663426339	0.9645970368		
	Tank w/ Diesel Pump	0.9996220316	0.9985312499		
	Tank w/ Electric Pump	0.9996954766	0.9986061399		
Tank w/ Diesel Pump	Well w/ Electric Pump	0.9995058577	0.9980935990	0.9916171694	0.9936004037
	Open Water w/ Electric Pump	0.9996227052	0.9985337871		
	Tank w/ Diesel Pump	0.9999957630	0.9999391717		
	Tank w/ Electric Pump	0.9999965863	0.9999422733		

Primary	Secondary	Class A	Class B2	Class C1	Class C2
Elevated Tank	Well w/ Diesel Pump	0.9999859364	0.9998399091	0.9987580786	0.9999959970
	Well w/ Electric Pump	0.9999859542	0.9998401072		
	Open Water w/ Diesel Pump	0.9999892564	0.9998768136		
	Open Water w/ Electric Pump	0.9999892755	0.9998770264		
	Tank w/ Diesel Pump	0.9999998796	0.9999948982		
	Tank w/ Electric Pump	0.9999999030	0.9999951584		
	Elevated Tank	0.9999999966	0.9999995721		

Appendix E Distribution Reliability

Table 36 Distribution Reliability for Australian supplies below 50,000 connection. NWC (2009)

Location	Customer interruption frequency-water (per 1,000 properties)	Water main breaks (per 100 km of water main)	Length of water mains (km)	Average duration of an unplanned interruption-water (minutes)	Expected Shutdowns per year (minutes)	Frequency of Shutdowns per year in minutes	Reliability
East Gippsland	20	211.0	9.0	838	121.5	26	0.999951
GWMWater	30	226.5	54.0	1,245	81.3	18	0.999965
Lower Murray Water	30	236.1	51.0	873	76.5	18	0.999966
North East Water	43	47.9	17.0	1,406	121.3	6	0.999989
P&W - Darwin	47	119.9	41.5	1,258	56.6	7	0.999987
Port Macquarie-Hastings	29	12.0	4.0	753	180.0	2	0.999996
Wannon Water	40	69.9	15.1	1,754	88.2	6	0.999988
WC - Mandurah	36	82.1	5.9	758	107.0	9	0.999983
Western Water	46	165.6	22.6	1,645	101.6	17	0.999968
Aqwest Bunbury	15	0.2	13.0	339	30.0	0.0	1.000000
Busselton Water	10	78.0	10.9	256	46.2	3.6	0.999993
Dubbo	16	26.3	5.0	451	112.0	3.0	0.999994
P&W - Alice Springs	12	286.3	56.9	378	117.7	33.8	0.999936
South Gippsland	17	251.7	93.0	615	115.8	29.2	0.999944
WC - Albany	14	138.6	12.9	434	97.0	13.5	0.999974
WC - Geraldton (W)	15	64.5	52.4	505	56.0	3.6	0.999993
WC - Kal-Boulder (W)	14	62.3	21.1	275	158.0	9.9	0.999981
Westernport Water	14	393.7	-	368	78.1	30.9	0.999941